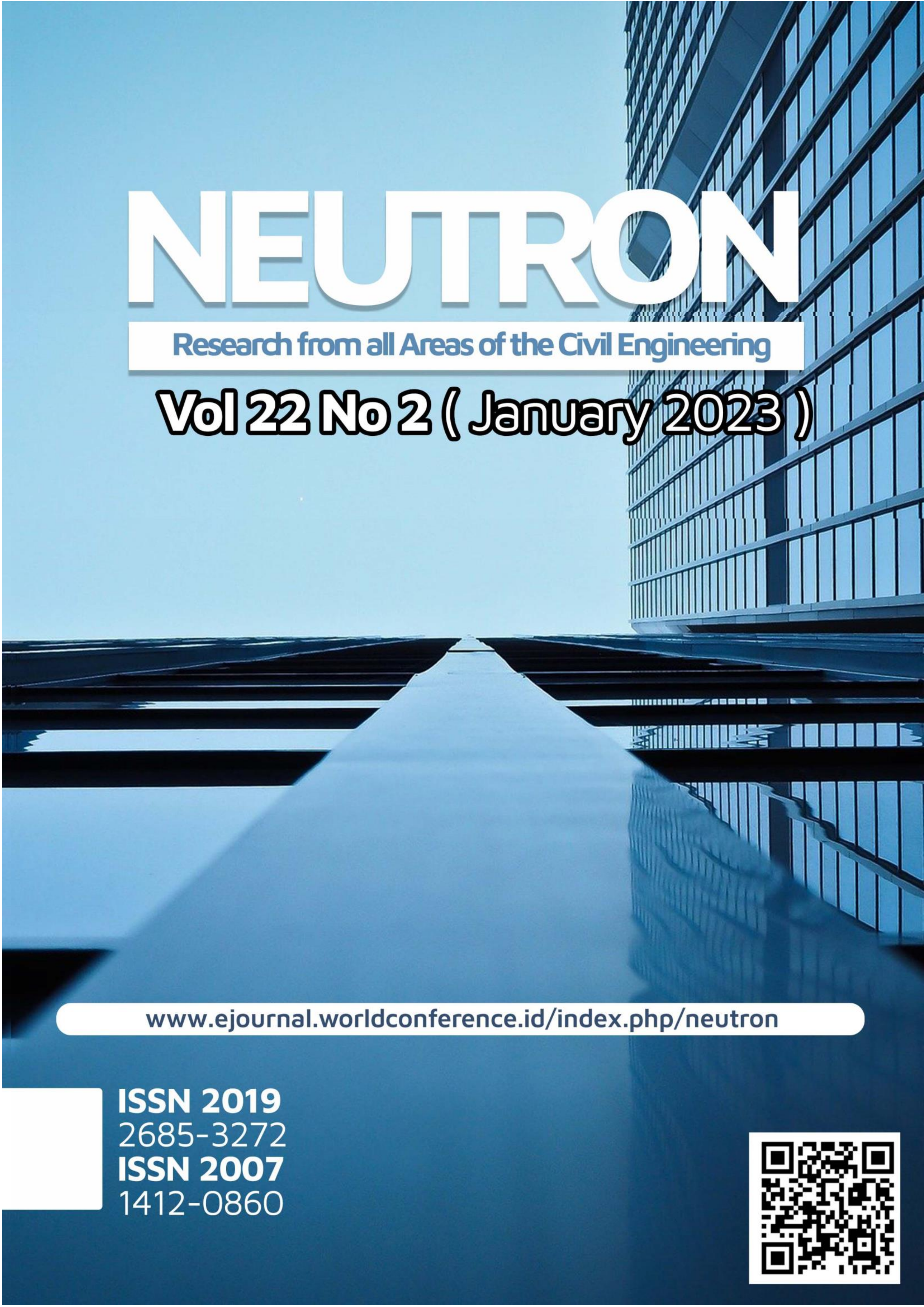

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Modeling of Flow Rate at Sukkur Barrage using Artificial Neural Networks (ANNs)

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Abstract

Modeling of flow discharge plays a significant role in effective planning, sustainable usage, development, and management of water resources in short (hourly) and long-term (monthly) temporal categories. Since the inception of managing water resources, various techniques such as conceptual, metric, and physical models have been introduced all of these require a large amount of data, labor, and expense to be incorporated to obtain reliable results, due to which Artificial Intelligence methods were introduced that require less amount of data, time, expense and as well as experience to model flow discharge. In this research study, an attempt was made by employing two different artificial neural network techniques feedforward neural networks (FFNN), and time-lagged neural networks (TLNN) to model and predict the river flow discharge at daily and monthly timescale. 2010 and 503 no. of observations were used for model calibration and validation in daily time scale while 557 and 139 observations were used in monthly timescale. The result of the study revealed that the FFNN modeling approach has captured the daily and monthly stream flow variability very well than the TLNN model with R2 of 0.91 on the daily and 0.71 on the monthly time scale while R2 for the TLNN model was 0.79, and 0.34 for daily and monthly timescale. This indicates that the FFNN technique requires less no. of observations and is more reliable than TLNN and can be used to model river flow.

Keywords :
Artificial Neural Networks, Feedforward Neural Networks, Machine Learning, River Flow Discharge, Stream Flow Prediction, Sukkur Barrage, Timeseries Analysis, Time-lagged Neural Networks

1. Introduction

Rivers serve as a significant source of fresh water for living beings (Bisoyi et al., 2019). Therefore modeling of river flow is of prime concern to guarantee the sustainable development, effective planning, development, and management of water resources (Parisouj et al., 2020). In an irrigation system, information on the flow rate (Q) is vital for various purposes such as; planning, designing, and operating hydraulic structures, managing water distribution among stakeholders, assessing extreme flood events, adopting mitigation measures, and drought warnings and reservoir operation (Hussain & Khan, 2020; Yaseen et al., 2015).

There are two broad modeling approaches used to model river flow: physically based models and data-driven models (Masselot et al., 2016). Physically-based models require a large number of data, finance, experience, complex mathematical tools, and human effort to build, calibrate, and validate the models(Khazaee Poul et al., 2019), while data-driven models are self-adaptive, more easily developed, and capable of capturing non-linear processes numerically using limited data inputs with no knowledge of the underlying physical processes involved (Noori & Kalin, 2016; Wu & Prasad, 2017; Yaseen et al., 2015). Since 1970, many data-driven approaches including linear regression (LR), multi-linear regression (MLR), moving average (MA), and auto-regressive moving average (ARMA) have been used for the modeling of river flow (Yaseen et al., 2015). These models were less accurate, linear, and unable to capture non-linearity b/w hydrological data (Melesse et al., 2011), so researchers have concentrated on developing innovative prediction models which will need not only to be inexpensive but also to be highly accurate and reliable (Hussain & Khan, 2020).

An artificial neural network is an innovative powerful tool, which is inexpensive, accurate, and reliable in terms of capturing non-linear temporal variation in data (Cheng et al., 2020), it is also capable of handling and analyzing large datasets (Buyukyildiz & Kumcu, 2017), so to extract/ recognize the patterns in the historical data, applies those to predict future scenarios (Khazaee Poul et al., 2019). The architecture of an Artificial Neural Network is composed of an input layer, a hidden layer, and an output layer. These three layers contain neurons that are connected through the connecting link called nodes. Compared to conventional methods, artificial intelligence methods can address the noise dataset in a non-linear dynamic system (Adedigba et al., 2017). Artificial Neural Networks have been utilized in various applications of hydrology engineering. Artificial neural networks have been utilized for solving various hydrological problems that include; groundwater simulation (Taormina et al., 2012), modeling of rainfall-runoff relationship (Patel & Joshi, 2017), prediction of river flow (Hussain et al., 2020), and simulation of water quality (Kadam et al., 2019). Various investigations were carried out from time to time by several researchers to reach the final efficient modeling approach that can not only save time but also provide reliable prediction results, some of those studies are described below;

(Büyükşahin & Ertekin, 2019) utilized a conventional modeling technique (ARIMA), and an artificial neural network modeling technique on a monthly stream flow time series. The result revealed that the ANN model performed better than ARIMA. (Ghorbani et al., 2016) carried out a comparative study, in which flow discharge was modeled by (ANN), (MLR) (RC), and (SVM), and their performances were compared with each other. The result revealed that values predicted by ANN, and SVM were more accurate and reliable than those by RC and MLR.

(Khazaee Poul et al., 2019) conducted a comparative study, in which monthly flow discharge was modeled and predicted by ANN, ANFIS, KNN, and MLR. The result of the study revealed that all three methods act significantly better than the MLR models. The MLR model tried to fit a linear relationship between the inputs and outputs. MLR was not able to estimate the monthly flows precisely enough. (Hussain & Khan, 2020) used three different machine learning modeling approaches; artificial neural network (ANN), support vector regression (SVR), and random forest (RF) to forecast Hunza River flow. The results found that ANN & RF performed slightly better than SVR. The main objective of the research study was to investigate the forecasting capabilities of the ANNs model for long-lead time on two-time scales.

2. Methodology

2.1. Study Area

Sukkur barrage is located at a longitude of 68.8471°, and a latitude of 27.6733° N, and was constructed on the River Indus in 1932 with the design discharge of 1.5 million cusecs for diverting water to canals at its upstream side with the purpose to ensure sustainable water provision to mankind for irrigating their lands, fulfilling anthropogenic and industrial water demands, flood control, and regulating the flow in the river.

The land that is cultivated by off-taking canals from the Sukkur barrage gives sufficient agricultural productivity to fulfill not only the need for rural and urban livelihoods in Sindh province but also to support the economy of our country. (Sindh barrage improvement project, 2018).

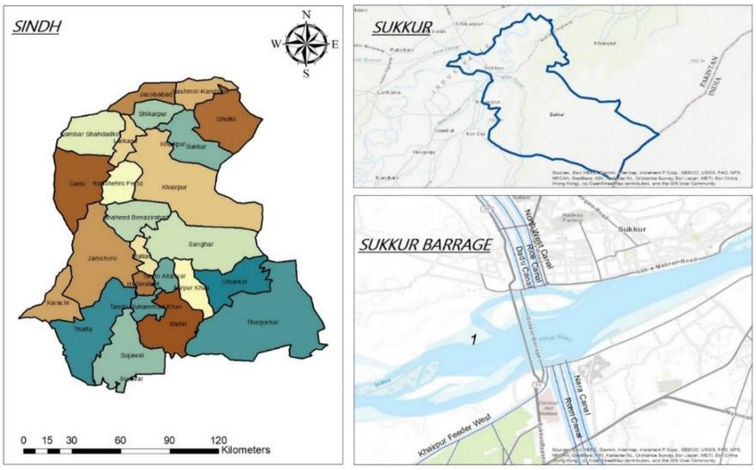

Figure 1. Shows the map of the study area of Sukkur Barrage

2.2. Steps of Model Development

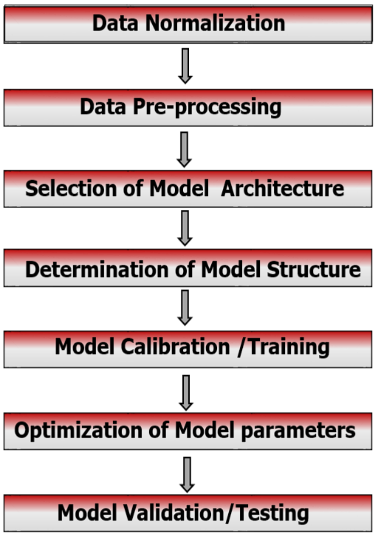

Figure 2. Shows the step-by-step procedure used for the model development.

2.2.1. Data Collection

In this research study, time series of flow discharge from Jan 2013 to December 2019 was acquired from the Department of Sindh Irrigation and Drainage Authority, Sukkur Barrage Division (SIDA).

2.2.2. Data Normalization

The input data were processed and normalized by the below equation to prevent the model from being dominated by variables with large values.

$$QS \frac{Qi - Qmin}{Qmax - Qmin} \qquad (1)$$

Where,
Qs = normalized value of data
Qi = observed value of data
Q min = minimum value of data
Q max = maximum value of data

2.2.3. Selection of Model Network

Among various well-known available artificial neural networks; two modeling approaches Multilayer Perceptron (MLP), and Time-Lagged Neural Networks (TLNN) were selected.

2.2.4. Determination of Model Structure

A model structure contains 11 input neurons, 1 hidden layer, and 24 numbers of hidden neurons, log sigmoid as a non-linear activation function was selected and the flow of data was chosen as straight/feedforward

$$af = \sum_{i=1}^{n} XiWi \qquad (2)$$

2.2.5. Data Splitting

The data was divided into two parts (70% and 30%); one for training, and the other part for testing/validating the developed model (Araghinejad, 2014).

2.2.6. Model Calibration

In the training phase initially, random weights were assigned to the data inputs by a non-linear activation function (eq. 3.) to adjust them time by time wherever needed to minimize the error between observed and predicted data.

$$f(af) = \frac{1}{(1+e^{-af})} \qquad (3)$$

2.2.7. Model Validation

After the development, and calibration of the model, validation was performed by using different statistical indices that include; Coefficient of Regression (R2), Mean Absolute Error (MAE), and Root Mean Squared Error (RMSE) to test the reliability and predictive performance of the developed model.

$$R2=1 - \frac{\sum_{i=1}^{n}(OBS.i - PRE.i)^2}{\sum_{i=1}^{n}(OBS.i - \overline{OBS})^2} \qquad (4)$$

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$$\text{MAE} = \frac{1}{n}\sum_{i=1}^{n}(Qp - Qo)^2 \quad (5)$$

$$\text{RMSE} = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(Qp - Qo)^2} \quad (6)$$

3. Results and Discussion

In this research study, Two Different types of ANN models; Multilayer Perceptron (MLP), and Time-Lagged Neural Network (TLNN) models have been selected and used for Flow Rate Prediction at daily and monthly timescale.

Table 1. shows the statistical Distribution of the Daily and Monthly datasets

Timescale	Total no. of Observations	Observations used for Training	Observations used for Testing	Standard Deviation (sd)	Max (ft3/sec)	Min (ft3/sec)	Mean (ft3/sec)
Daily	2513	2010	503	83208	709316	8980	75125
Monthly	696	557	139	121606	886524	7798	102460

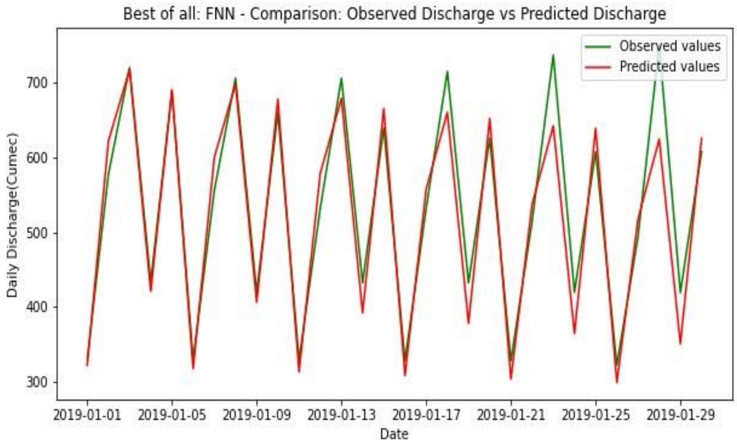

Figure 3. shows the observed and predicted values for January 2019, FNN model captured the daily variability well up to the 17th of January, then under-predicted the peak flows up to the end of the month.

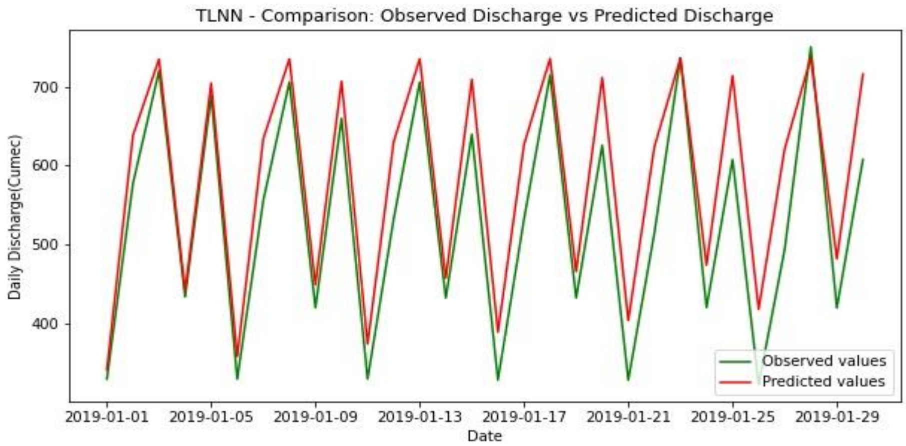

Figure 4. shows the observed, and predicted values by Time-Lagged Neural Network (TLNN), TLNN predicted the base as well as peak well up to the 23 days of the month.

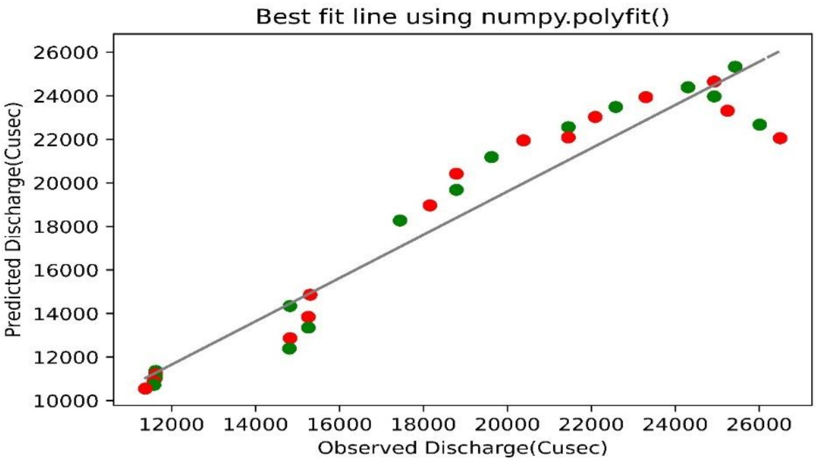

Figure 5. Scatterplot showing the linear relation between observ-ed and predicted discharge

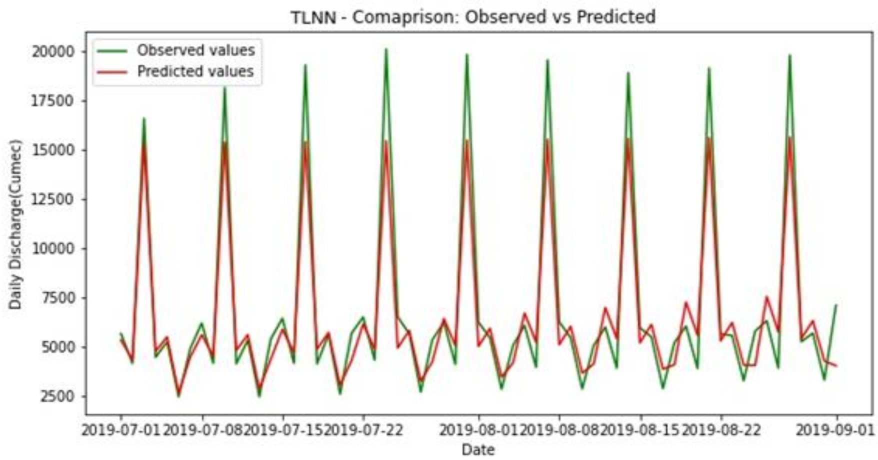

Figure 6. shows the observed and predicted values for the month of July-August 2019, TLNN model captured the base flow well up to July and later failed to capture base flow as well as peak flows

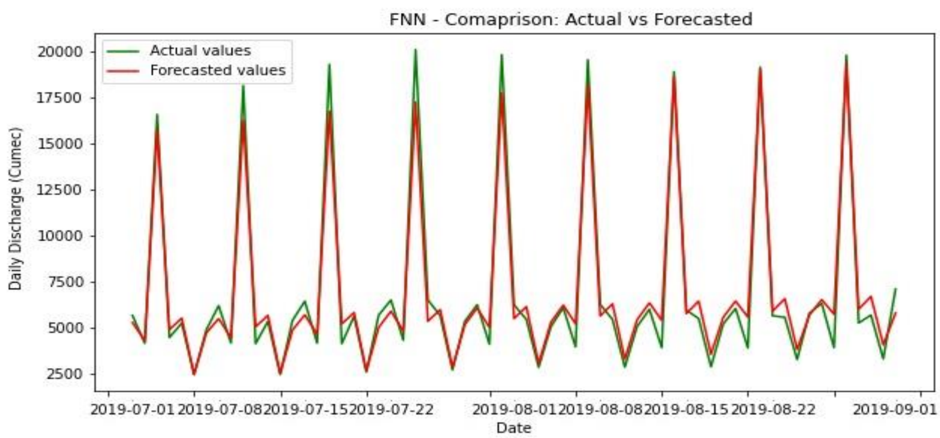

Figure 7. shows the Observed and Predicted values by FNN, which performed very well in capturing base and peak flows but slightly under-predicted the peak flows at the middle of the data points

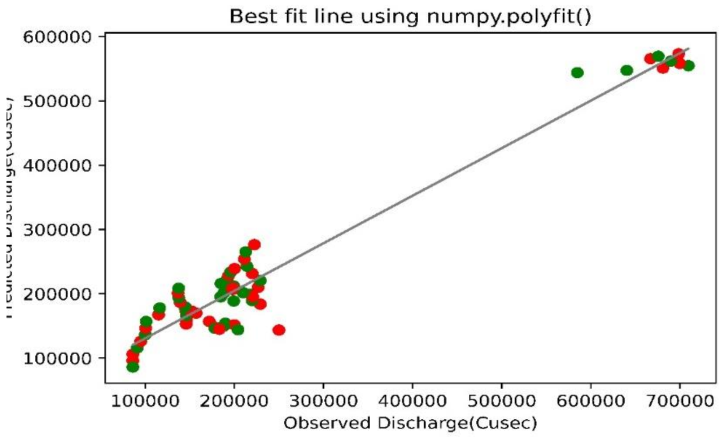

Figure 8. Scatterplot shows the relationship between observed and predicted discharge

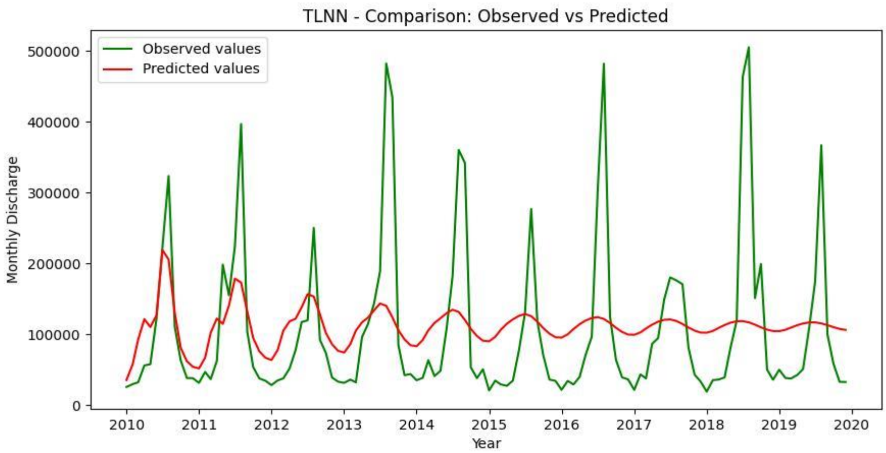

Figure 9. shows the Monthly Observed and Predicted values by TLNN from 2010-2020, which performed worst and failed to capture the base as well as peak flows

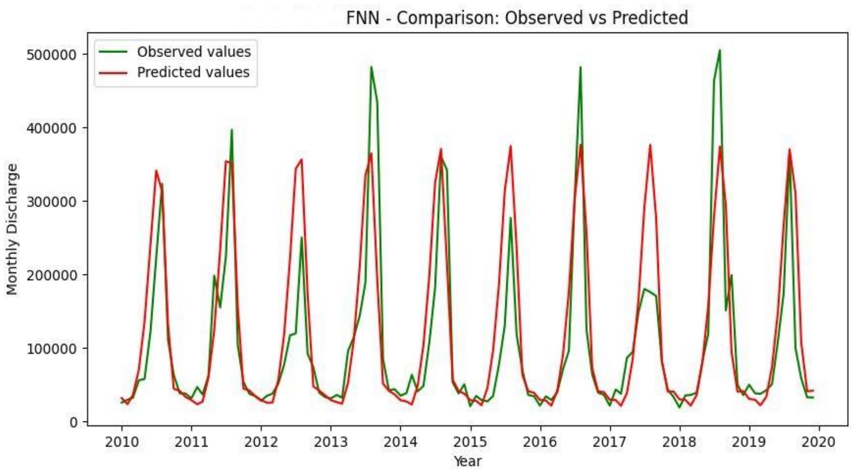

Figure 10. shows the Monthly Observed and Predicted values, FFNN captured the base flow very well at all points, it also predicted peak flows well at some points and slightly under-predicted the peaks in 2013, 2016, and 2018.

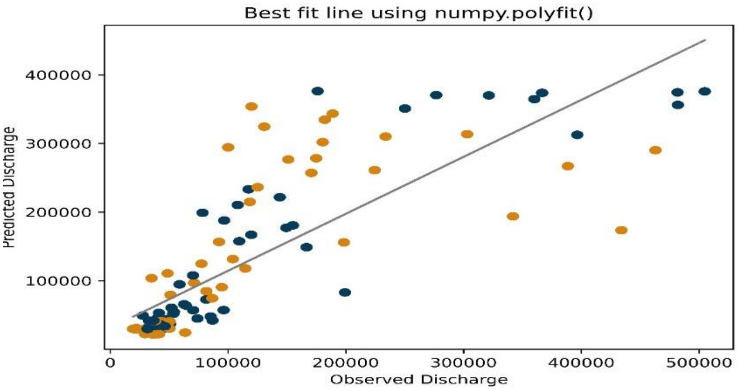

Figure 11. Scatterplot shows the distribution of observed and predicted discharge on a monthly timescale.

Table 2. shows the predictive performance of the developed models based on MAE, RMSE, and R2

Sr, No.	Timescale	Model	MAE (%)	RMSE (%)	R2 (%)
1	Daily (low Flows)	FNN	0.46	0.79	0.91
2	Daily (low Flows)	TLNN	2.01	2.63	0.79
3	Daily (high Flows)	FNN	1.74	2.31	0.96
4	Daily (high Flows)	TLNN	3.21	4.69	0.89
5	Monthly	FNN	17.3	28.29	0.76
6	Monthly	TLNN	200	289	0.33

4. Conclusion

Two ANN modeling approaches have been employed for the prediction of one step ahead at daily and monthly timescales. In the daily timescale among these 2 models, FFNN has performed better for low flows as well as for High flows, while TLNN has captured the daily variability less significantly than FFNN. In the monthly timescale, FFNN has captured base and peak flows very well, but TLNN completely failed to capture the monthly flow discharges, the reason for this is that in this case study, the number of observed monthly datasets used was much smaller than that of daily datasets. TLNN cannot be used where there is less availability of no. of observations for training and testing the model. In addition, it is noted that the number of epochs required for FFNN model convergence was much smaller than the TLNN model. This indicates that the FFNN model is more suitable and can extract nonlinearity characteristics of data more efficiently in comparison to the TLNN model.

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Assessment of Groundwater Quality using Water Quality Index, and Geo-Spatial Tools

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Abstract

Groundwater is considered a significant component of valuable freshwater resources for the living beings living in the arid and semi-arid regions of Pakistan, factors such as climate change, landfill deposits, application of fertilizers and pesticides on agricultural lands, leakages from septic tanks, industrial effluents, and urbanization jeopardizing the groundwater quality that makes its timely assessment necessary for human survival to protect people from water-borne diseases. This research study was carried out to analyze the 13 physicochemical parameters in the 38 groundwater samples that were taken from nearby different locations of Akram, Pinyari, and Phuleli canal within the jurisdiction of district Hyderabad. 26 samples were taken from hand pumps, electric motors and 12 samples were taken from tube wells to investigate the variability in groundwater quality. The water quality index and kelly's ratio, magnesium hazard, residual sodium carbonate, sodium absorption ratio, soluble sodium percentage, and permeability index were used to assess the suitability of groundwater for drinking, and agricultural purposes. The result of the study revealed that alkalinity, bicarbonates, carbonates, magnesium, PH, potassium, and sodium were within the permissible limit of WHO Standards in all the samples, while the concentration of calcium was crossing the permissible limit only in one sample. Moreover, the availability of Chloride (Cl) was found in 16 samples that were above the limit ranges from 274.9 to 2549, High concentration of EC was found in 14 samples than the permissible limit having values from 2212 to 8360 (P26), and Total Hardness (TH) were found only in 6 samples slightly high from the permissible limit ranges from 509 to 651, and TDS were present in excessive amounts than the allowable limit from 1100 to 4180 mg/l (P26) in 10 samples. Considering the Water Quality Index (WQI) it was observed that 8 sample falls in the good category, 2 samples in the poor category, 11 samples in the very poor category, and 17 samples in the unsuitable category of water quality.

Keywords:
Agricultural, Akram Wah, Drinking, GIS, Groundwater, Hyderabad, Phuleli Canal, Physicochemical Parameters, Pinyari Canal, Water Quality, WQI,.

1. Introduction

Groundwater is considered an natural vital resource on the earth that is utilized by living beings for agricultural, domestic, drinking, and industrial purposes (Shahab et al., 2018). Fresh Groundwater is an essential resource available on the earth that every human being is utilizing for agricultural, drinking, domestic as well as industrial purposes. In a country like Pakistan, the occurrence of rainfall patterns is uneven due to its arid, and semi-arid climatic nature also the less availability of surface water has enforced to increase the dependence on groundwater. In Sindh, only 10% of the area, fresh groundwater is available (Alamgir et al., 2016).

In Pakistan, Hyderabad is ranked 2nd among the largest cities in Sindh Province. The shift of people from rural-based to urban-based areas, lack of proper sewer systems, and effluents discharged from the industrial sector in the city have triggered groundwater contamination (Silva et al., 2021).

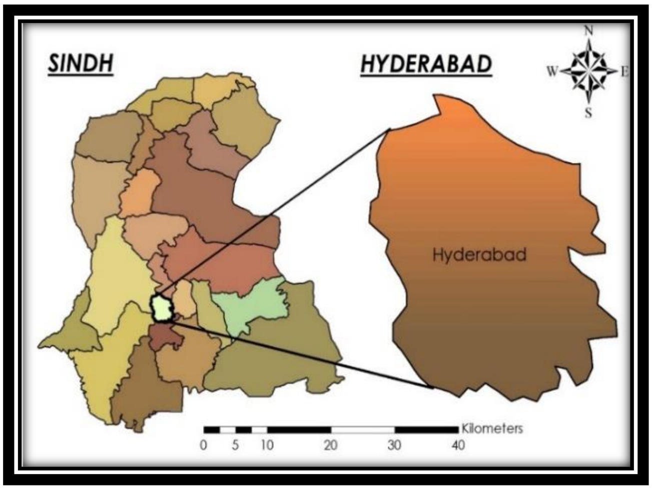
Since this natural resource is dynamic, its contamination may spread in time and space which needs to be assessed . Due to the variable nature of groundwater, its contamination may spread over time, and space necessitates immediate assessment to protect this vital resource from contamination (Talib et al., 2019). Around 70-80% of all human diseases are caused by contaminated water(Maity et al., 2020). In rural areas majority of people still totally rely on untreated groundwater, it is vital to find a solution because safe access to drinking water is a fundamental human right (Raza et al., 2017). The formation of balanced socioeconomic development is based on the quantity, quality, and safe accessibility of water resources, and its optimal use cannot be maintained without an evaluation of its quality. In 1965, Horton established the water quality index (WQI), and later on, Brown 1972 made some improvements to it, further modifications were performed by a Scottish Scientist Deininger in 1975 at the Scottish development department (Singh et al., 2022). The WQI is a mathematical equation that was developed to simplify the communication of information to all stakeholders by converting a significant amount of water quality data into a single number. Various research studies were conducted by researchers, some of them are discussed below;

(Singh et al., 2022) determined the groundwater quality using WQI and GIS applications in 89 small villages. Twelve water quality parameters were selected for the study area. Results revealed that the WQI ranged from 71 to 447, result also identified 68% of the groundwater of the study area had poor water quality while the remaining 32% is considered as good quality water that is allowable to consume. Fluoride, TDS, and pH were found to be excessive amounts than the allowable limit. The study also establishes that a higher number of the population is still consuming the water, despite the fact it is highly vulnerable and is prone to many water-borne diseases.

(Qureshi et al., 2021) conducted the research study in Tandojam city of Sindh, in which groundwater quality parameters of 30 samples were analyzed. The results of the study revealed the availability of alkalinity, chloride, EC, nitrate, pH, TDS, TH, turbidity, and sulfate in most of the samples in a higher concentration than the permissible guidelines. (Lanjwani et al., 2020) analyzed the parameters of groundwater quality of Larkana, Sindh. The result concluded that 12 samples among 25 samples were considered as not suitable for drinking purposes due to the high presence of chloride, EC, fluoride, and TDS. (Solangi et al., 2019) have evaluated the quality of groundwater in the sujawal district in the province of Sindh, a total of 94 samples were taken through different locations of the study area. Results of the study revealed that 77.66% of samples were poor to very poor quality, 13.8% of samples were considered unsuitable for drinking purposes while the remaining 8.51% of samples were of excellent to good quality based on the water quality index (WQI) and synthetic pollution index (SPI) indicated that 45.83% of samples were slight to moderately polluted, 20% of samples were highly polluted while 18% were suitable and 16% were considered as unsuitable. The authors also revealed that the quality of most groundwater samples exceeded the permissible limit prescribed by WHO and suggest that using that water may have a high risk and can cause several diseases.

2. Methodology

2.1. Study Area



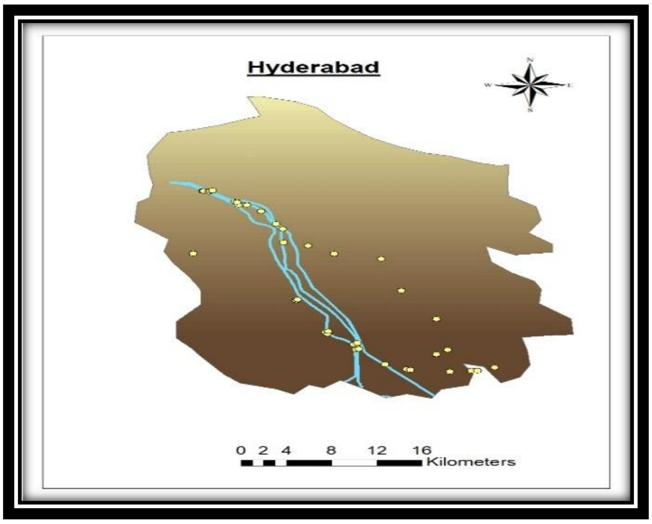


Figure 1. shows the map of the study area of Hyderabad district.

2.2. Collection of Geo-Referenced Groundwater Samples

38 samples were collected along the canals through hand pumps, electric motors, and tube wells in 1-liter polyethylene bottles. Before collection of samples, bottles were properly washed (first with tap water, then with detergent, and rinsed with distilled water for removal of any contamination. While collecting GW samples, the depth of groundwater and GPS coordinates were also recorded. Water samples were collected after the purging process. The purging of a 30 ft depth dug well needs 30 strokes of a hand pump. Standard SoPs were followed for the collection of water samples and their transportation to USPCAS-W.

2.3. Assessment, and Characterization of the Groundwater Quality for Drinking, and Agricultural Purposes

The samples were assessed and analyzed for the below-listed parameters and water quality was characterized and categorized as per WHO standards.

Table 1. Shows the Physicochemical Parameters Analyzed in the Groundwater Samples.

Alkanity	Chloride (Cl)	Potassium (K)
Bicarbonates(HCO3)	Electrical Conductivity (Ec)	Power of Hydrogen (PH)
Carbonates (CO3)	Hardness	Sodium (Na)
Calium (Ca)	Magnesium (Mg)	Total Dissolved Solids (TDS)

2.4. Evaluation of the Suitability of Groundwater Based on the Following Equation for Drinking and Irrigation Purposes

$$\mathrm{WQI} = \sum \mathrm{Wi} \times \mathrm{Qi} \quad (1)$$

$$\mathrm{SP} = \frac{(\mathrm{Sodium+Potassium})}{(\mathrm{Calcium+Magnesium+Sodium+Potassium})} \times 100 \quad (2)$$

$$SAR = \frac{\mathrm{Sodium}}{\sqrt{(\mathrm{Calcium+Magnisium})/2}} \quad (3)$$

$$\mathrm{RSC} = \mathrm{Bicarbonate} + \mathrm{Carbonate} - (\mathrm{Calcium} + \mathrm{Magnesium}) \quad (4)$$

$$MH = \frac{Magnesium}{Calcium+Magnesium} \times 100 \quad (5)$$

$$\mathrm{PI} = \frac{\mathrm{Sodium}+\sqrt{\mathrm{Bicarbonate}}}{\mathrm{Calcium+Magnesium}} \times 100 \quad (6)$$

$$\mathrm{R} = \frac{\mathrm{Sodium}}{\mathrm{calcium+Magnesium}} \quad (7)$$

Table 2. Shows the Different Indices Used to Analyze the Suitability of Groundwater Samples.

Parameters	Ranges	Water Classes
SAR	<10	Excellent
	10-18	Good
	18-26	Fair
	>26	Poor
RSC	<1.25	Safe
	>1.25	Unsuitable

SSP	<20	Excellent
	20-40	Good
	40-60	Permissible
	60-80	Doubtful
	>80	Unsuitable
KR	<1	Safe
	>1	Unsuitable
MH	<50	Good
	>50	Unsuitable
PI	<25	Good
	25-75	Suitable
	>75	Unsuitable

2.5. Preparation of Groundwater Quality Maps and Infer Guidelines Regarding the Water Quality Status in District Hyderabad

GIS maps of each of the water quality parameters were prepared using the Interpolation Inverse Distance Weighted (IDW) method in ArcGIS 10.8 through these maps, a conclusion about the water quality of the Hyderabad district was drawn.

3. Results and Discussion

Groundwater suitability for drinking and irrigation purposes is based on the water quality index and irrigation indices. The results and discussion are made as follows.

Table no. 03, shows the presence of physicochemical concentration in groundwater samples.

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ID	Alkal.	HCO3 (mg/l)	CO3 (mg/l)	Ca (mg/l)	Cl (mg/l)	EC µs/cm	TH (mg/l)	Mg (mg/l)	K (mg/l)	PH	Na (mg/l)	TDS
P1	5	5	0	47.03.00	119.09.00	1072	326.07.00	50.09.00	12.02	07.17	97.01.00	536
P2	5	5	0	44.04.00	159.09.00	1252	265.03.00	37.07.00	04.07	07.21	75.05.00	626
P3	5	5	0	46.07.00	49.09.00	575	165	12	06.04	07.24	26.02.00	288
P4	5	5	0	141.01.00	379.08.00	2212	650.03.00	81.07.00	16.06	07.46	163.01.00	1106
P5	5	5	0	144.09.00	469.08.00	2620	649.02.00	95	05.08	07.31	205.06.00	1309
P6	02.05	02.05	0	96.01.00	349.08.00	2400	491.09.00	61.06.00	05.08	07.29	135.03.00	1200
P7	5	5	0	54	99.09.00	1374	293	38.06.00	10.06	07.13	75.06.00	687
P8	02.05	02.05	0	96.02.00	224.09.00	962	376	33.04.00	06.05	07.26	77.02.00	480
P9	5	5	0	51.07.00	799.07.00	4120	292	39.08.00	05.07	07.51	224	2060
P10	5	5	0	81	174.09.00	1420	380.04.00	43.06.00	06.08	07.29	124.08.00	710
P11	5	5	0	74.07.00	149.09.00	1471	388.06.00	49.04.00	07.04	07.02	158.02.00	735
P12	5	5	0	139.06.00	399.08.00	2260	512.01.00	40.03.00	05.08	07.14	167.05.00	1133
P13	5	5	0	13.06	49.09.00	1094	93.09.00	14.06	02.04	0,3430556	186.08.00	547
P14	5	5	0	70.01.00	199.09.00	850	288.03.00	27.08.00	05.02	07.34	47.02.00	425
P15	5	5	0	72	474.08.00	2790	344.04.00	40.03.00	06.03	07.49	222	1399
P16	02.05	02.05	0	182.07.00	899.07.00	4170	651.09.00	56.08.00	10.07	07.17	214.03.00	2080
P17	02.05	02.05	0	183.06.00	249.09.00	1575	648	54.09.00	11.01	07.19	224.02.00	787
P18	02.05	02.05	0	61.07.00	174.09.00	960	223.09.00	17.02	04.07	07.25	45.06.00	479
P19	5	5	0	62.02.00	174.09.00	1358	256.02.00	24.08.00	04.02	07.35	146.06.00	679
P20	5	5	0	51.02.00	174.09.00	845	180.01.00	12.09	06.01	07.47	69.03.00	423
P21	5	5	0	49.09.00	224.09.00	1210	244.07.00	29.04.00	04.06	07.42	87.05.00	605
P22	5	5	0	50.01.00	324.08.00	1540	225	24.05.00	08.05	07.06	136.05.00	770
P23	5	5	0	32.03.00	99.09.00	653	168.04.00	21.05	04.01	07.49	39.01.00	326
P24	02.05	02.05	0	30.01.00	974.06.00	3880	222.08.00	36	4	07.57	214.01.00	1940
P25	5	5	0	30	649.07.00	3280	223.03.00	36.02.00	03.09	0,3388889	214	1646
P26	02.05	02.05	0	325.03.00	2549.02.00	8370	647.01.00	106.01.00	15.07	0,3597222	213.03.00	4180
P27	5	5	0	85.01.00	49.09.00	998	326.02.00	28	06.08	07.41	133	499
P28	5	5	0	94.05.00	474.08.00	1726	509.03.00	66.08.00	06.05	07.41	90.07.00	864
P29	5	5	0	131.02.00	349.08.00	1962	528.04.00	49.03.00	08.08	07.47	213	979
P30	5	5	0	130	274.09.00	1681	528.02.00	50	08.06	07.51	212.08.00	842
P31	5	5	0	112.09.00	74.09.00	1044	429.04.00	36.03.00	05.03	07.31	80.08.00	522
P32	02.05	02.05	0	43.04.00	449.08.00	1791	238.06.00	31.08.00	06.06	07.51	214.05.00	894
P33	02.05	02.05	0	50.05.00	99.09.00	870	207.09.00	20.01	05.08	07.54	83.01.00	434
P34	5	5	0	49.09.00	74.09.00	673	182	14.02	06.03	07.55	64.06.00	335
P35	5	5	0	61.05.00	274.09.00	1230	305.05.00	37.02.00	09.06	07.33	127.05.00	615
P36	5	5	0	36.08.00	149.09.00	1265	206.02.00	27.09.00	05.08	07.38	98.05.00	632
P37	5	5	0	39.08.00	49.09.00	729	170.07.00	17.05	06.08	07.29	66.05.00	364
P38	5	5	0	89.07.00	124.09.00	937	617.09.00	96	06.08	07.54	212.01.00	468

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Table 04, shows the obtained values of various indices used for the suitability of groundwater for irrigation purposes.

Sr.No	Sample ID	SSP	SAR	RSC	MH	PI	KR
1	P1	52.7	13.9	-93.2	51.8	101.2	1
2	P2	49.4	11.8	-77.1	45.9	94.6	0.9
3	P3	35.7	4.8	-53.6	20.5	48.4	0.4
4	P4	44.6	15.5	-217.8	36.7	74.2	0.7
5	P5	46.8	18.8	-234.8	39.6	86.7	0.9
6	P6	47.2	15.2	-155.2	39.1	86.8	0.9
7	P7	48.2	11.1	-87.7	41.7	84.1	0.8
8	P8	39.2	9.6	-127.1	25.8	60.8	0.6
9	P9	71.5	33.1	-86.5	43.5	247.3	2.4
10	P10	51.4	15.8	-119.6	35	101.9	1
11	P11	57.2	20.1	-119.1	39.8	129.2	1.3
12	P12	49.1	17.7	-174.9	22.4	94.4	0.9
13	P13	87	49.7	-23.2	51.6	669.6	6.6
14	P14	34.8	6.7	-92.9	28.4	50.5	0.5
15	P15	67	29.6	-107.3	35.9	199.7	2
16	P16	48.4	19.6	-237	23.7	90.1	0.9
17	P17	49.7	20.5	-235.9	23	94.7	0.9
18	P18	38.9	7.3	-76.4	21.8	59.8	0.6
19	P19	63.4	22.2	-82	28.5	171.1	1.7
20	P20	54	12.2	-59.1	20.2	111.5	1.1
21	P21	53.7	13.9	-74.3	37	113.1	1.1
22	P22	66	22.4	-69.6	32.8	186	1.8
23	P23	44.6	7.5	-48.7	40	76.9	0.7
24	P24	76.7	37.2	-63.6	54.4	326.4	3.2
25	P25	76.7	37.2	-61.2	54.7	326.8	3.2
26	P26	34.7	14.5	-428.8	24.6	49.8	0.5
27	P27	55.3	17.7	-108.1	24.7	119.6	1.2
28	P28	37.6	10	-156.3	41.4	57.6	0.6
29	P29	55.1	22.4	-175.5	27.3	119.2	1.2
30	P30	55.1	22.4	-175	27.8	119.4	1.2
31	P31	36.6	9.4	-144.2	24.3	55.7	0.5
32	P32	74.6	35	-72.7	42.3	287.3	2.9
33	P33	55.8	14	-68.1	28.5	120.1	1.2
34	P34	52.5	11.4	-59.1	22.1	104.4	1
35	P35	58.2	18.2	-93.7	37.7	131.6	1.3
36	P36	61.7	17.3	-59.7	43.2	155.7	1.5
37	P37	56.1	12.4	-52.3	30.5	120	1.2
38	P38	54.1	22	-180.8	51.7	115.4	1.1

3.1. GIS Maps of Concentration in the Groundwater of the Study Area

3.1.1. Alkalinity and Bicarbonates

Alkalinity and bicarbonates concentration in all the samples were within the permissible limit with a minimum value of 2.5 mg/l and a maximum of 5 mg/l.

3.1.2. Calcium

In the current study area, the Calcium concentration ranges from 13 to 325 mg/l. Only 1 sample exceeds the permissible limit (200 mg/l).

3.1.3. Carbonate

Carbonate concentration in the study area was within the permissible limit (200 mg/l).

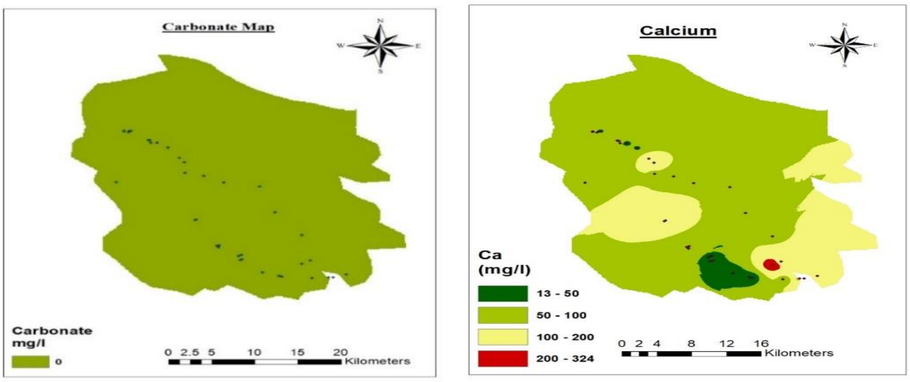


Figure 2 and 3. Show the behavior maps of calcium and carbonate.

3.1.4. Chloride

In the present study, the concentration ranges from 49.9 to 2549 mg/l. 16 samples are above the permissible limit (250 mg/l).

3.1.5. Electrical Conductivity

In the present study, the electrical conductivity varies from 576 to 8360 µS/cm. The results of 14 samples crossed the allowable limits of the WHO standard of 1500 µS/cm.

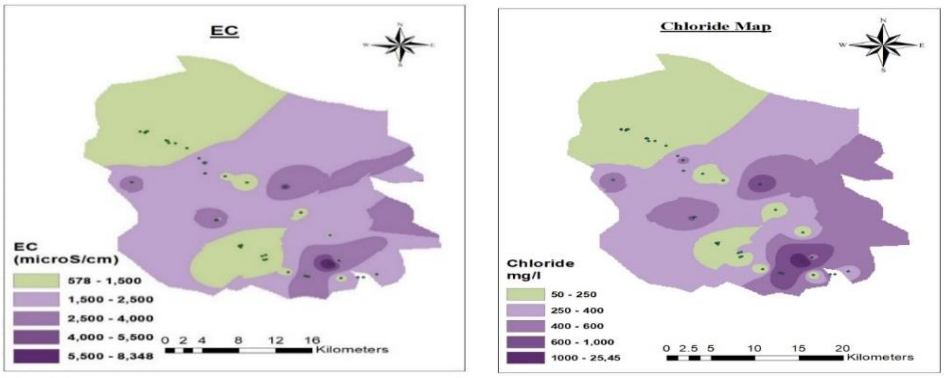


Figure 4 and 5. Show the spatial distribution maps of chloride and electrical conductivity

3.1.6. Hardness

In the current study, area hardness ranges from 94 to 651 mg/l. Only 8 samples contain above the permissible limit (500 mg/l).

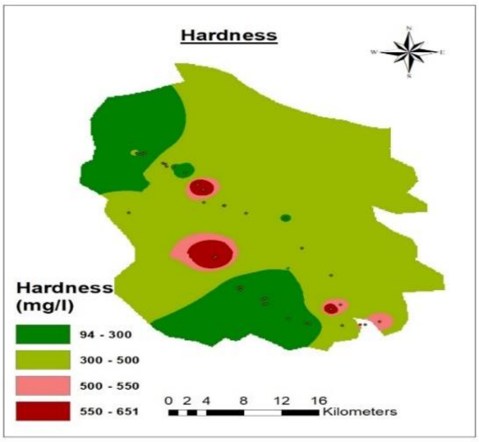


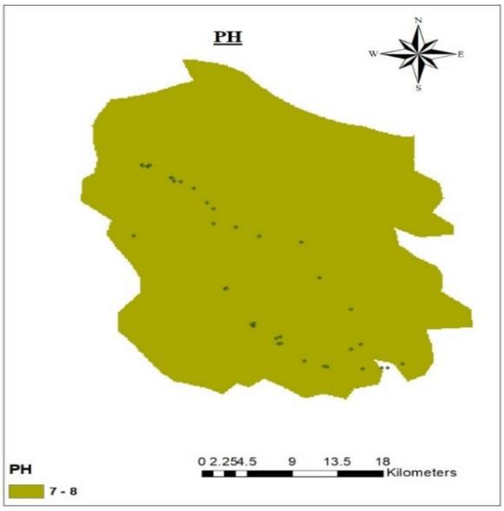
Figure 6, Show the spatial distribution map of hardness

3.1.7. Magnesium

In the study area, the concentration ranges from 12 to 106 mg/l. All sample lies within the permissible limit (150 mg/l). The higher concentration of Magnesium causes the Hardness of water.

3.1.8. pH

In the current study, the pH ranges between 7.13 (minimum) to 7.98 (maximum) with an average value of 7.45, the pH of all the samples was within the permissible limit (6.5–8.5) of WHO.



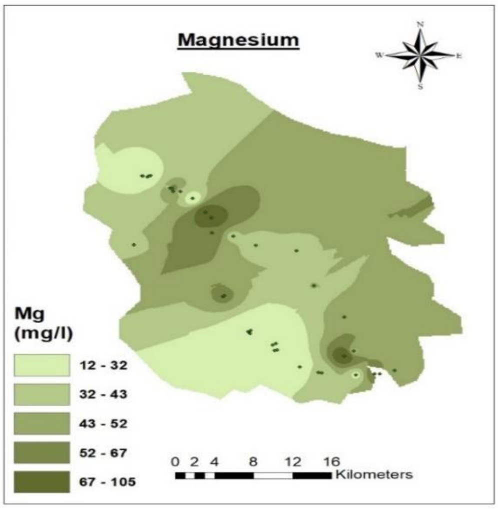


Figure 7 and 8, Show the spatial distribution maps of magnesium and Ph

3.1.9. Potassium

In the present study area, it varies from 2.42 to 16.58 mg/l. Only 2 samples are above the permissible limit which is (12 mg/l).

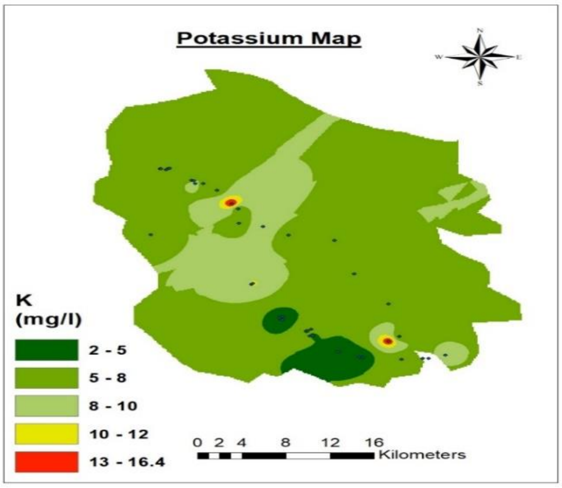


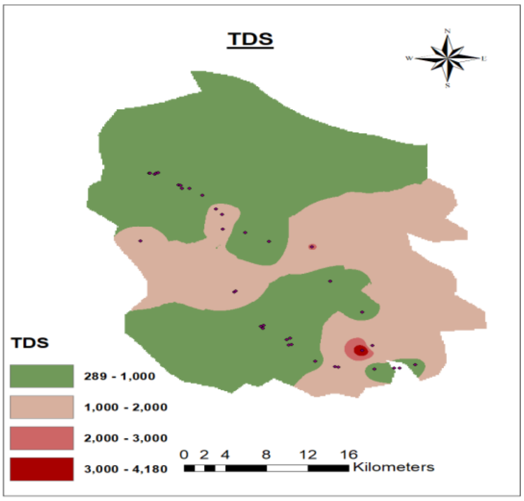
Fig 09, Show the spatial distribution map of Potassium

3.1.10. TDS

TDS in the study area ranged from 288 to 4180 mg/l. 10 samples crossed the allowable limits of the WHO, max permissible limit is 1000 mg/l.

3.1.11. Sodium

The permissible limit of sodium is 400 mg/l and in the current study area, it varies from 26.16 to 223 mg/l. All samples lie within ranges.



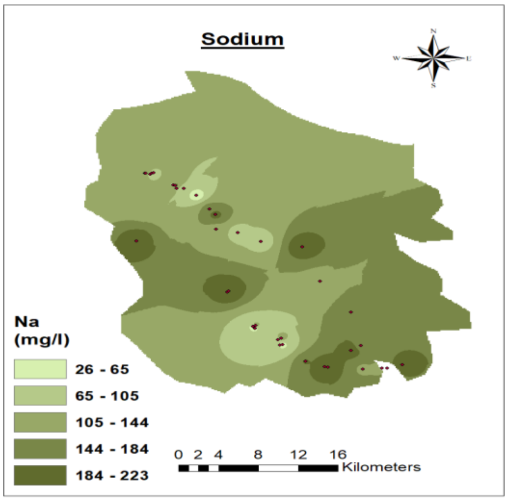


Fig 10 and 11, Show the spatial distribution maps of TDS and Sodium

4. Conclusion

In this research study, 38 groundwater samples were collected from different locations along the Akram, Pinyari, and Phuleli Canals for the assessment of groundwater quality, 12 physicochemical parameters were analyzed, among these 12 physicochemical parameters; alkalinity, bicarbonates, carbonates, magnesium, PH, potassium, and sodium were within the permissible limit of WHO Standards in all the samples, while the concentration of calcium was crossing the permissible limit only in one sample. Moreover, the availability of Chloride (Cl) was found in 16 samples that were above the limit ranges from 274.9 to 2549, High concentration of EC was found in 14 samples than the permissible limit having values from 2212 to 8360 (P26), and Total Hardness (TH) were found only in 6 samples slightly high from the permissible limit ranges from 509 to 651, and TDS were present in excessive amounts than the allowable limit from 1100 to 4180 mg/l (P26) in 10 samples. Considering the Water Quality Index (WQI) it was observed that 8 sample falls in the good category, 2 samples in the poor category, 11 samples in the very poor category, and 17 samples in the unsuitable category of water quality. In the present study, KR values range from 0.4 to 6.5. Out of 38 samples, except 19 samples, others had greater than 1 which means 50% of samples are suitable for irrigation. While SSP of 7 samples was good, 22 samples were permissible, 8 samples had doubtful, and 1 sample had the unsuitable quality of water for irrigation, which means few places are liable to sodium hazard. The MH value in collected samples varies between 20 to 60%. Only 5 samples were unsafe. The SAR values varied from 5 to 50%, In which 7 samples have excellent, 15 samples have good, 10 samples have fair and 6 samples had poor quality for irrigation purposes.

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Concrete Bridge Design Prt 80m Span Tension (Case Study of Kali Warkapi Bridge Km 49+250 Manokwari-West Papua)

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Abstract

Kali warkapi bridge km 49+250 Manokwari, West papua which connects several districts in West papua province stretches 80m based because the bridge previously decreased until it is no longer feasible to use, the type of steel frame bridge structure, then in this final project the design for the replacement of kali warkapi bridge using prestressed concrete structure type at this initial stage the analysis of exsisting conditions, planning of upper and lower bridge structures and calculations is carried out. Superstructure planning takes into account the loadsthat may occur, namely self-load, additional dead load, traffic load, wind load and earthquakeload. In planning the bridge is calculated using Ms.Excel.

Keyword :
Bridge Structure, Prestressed Concrete, Superstructure Planning

1. Introduction

1.1. Research Background

To improve living standards and promote the economy needed transportation facilities and infrastructure are very important functions, both land and sea transportation, and air. One of the roles of means that can support is the bridge. Prestressed concrete bridges are increasingly being used, because these bridges provide ease of implementation and have a lighter weight than other concrete bridges. This is because the weight of prestressed steel is much smaller than the amount of weight of ordinary concrete iron, and cannot be separated from the success high quality concrete (□c' ≥ 40 MPa) and high quality steel that has □y ≥ 1000 MPa. it will dilakuakn loading analysis and modeling calculation of building structures on concrete bridges prtegang as well as the calculation manually or using Ms.Excel as well as structural analysis of buildings on prestressed concrete bridges.

1.2. Problem Formulation

Based on the background above the problems to be examined are:

1. How is the modeling and analysis of the calculation of building structures on prestressed concrete bridges with width, and safe and optimal bridge spans?
2. how modeling analysis and structural work on prestressed concrete bridge with variation, width, and bridge span using Excel)?

1.3. Literature Review

Prestressed bridge planning kali suru pemalang Santosa et al. (2015) Suru bridge is located in the village of Suru,

Bantar bolang, Pemalang linking local andregional Kesesi Bantar Bolang route its span 144 meters above the river suru. Suru bridge replacement is based on conditions that exceed the design life of the bridge, steel truss bridge already rusty and the effective width of the bridge that does not meet the standards to serve the transportation needs.

Planning girder cross section i30meter span prestressed concrete bridge Nasution et al. (2020) Prestressed concrete structures are one of the preferred methods in building bridges today. Hal it considers longer usability and less maintenance. therefore the calculation analysis of prestressed concrete bridge girders for various spans becomes much needed. Prestressed concrete bridge building standards Director General of Highways Department of Public Works in 1993 has set the cross section size based on the length of the girder from 22 to 40 meters with an interval of every 3 meters so that not the entire span is available cross-sectional dimensions of the interval. At this writing the calculation of the girder cross section with a span of 30 meters will be analyzed to calculate the cableand tendon placement.

Prestressed concrete bridge structural design Hidayat & Chayati (2014) The structure of the bridge consists of lower and upper buildings. Building under the bridge is very dependent on the state of the ground, so it takes a soil investigation data that takes quite a long time and costs quite a lot. Given these reasons, the emphasis on the design of the structure of the bridge uses a lot of prestressed concrete structure system. In addition to the reason that the bridge spans tend to be long (30-90 m),

Redesigning Using Box Girder of the Upper Structure of Gajah Wong Bridge, Yogyakarta Hakim et al. (2013) The Gajah Wong bridge to be studied is located in road section of Selokan Mataram, connecting Gejayan Street and Seturan area, Sleman, Daerah Istimewa Yogyakarta Province. This bridge was built in order to increase economic and to support traffic activities in this area. Gajah Wong bridge has 40 m span. The designing of this bridge used I girder, and then would be done redesigning another form of prestressed-concrete bridge which is box girder. This redesign method being used is Bridge Management System (BMS).

Calculation of flyovers With box girder type prestressed concrete (prestressed concrete) to meet major alianyang road and soekarno-hatta road kubu raya regency (Prasetya et al., 2016). TProjection of traffic current is increasing of vehicle volume every year and for reducing conflict of perpendicular in junction of Jalan Mayor Alianyang and Jalan Sukarno-Hatta in Kubu Raya regency. Flyover become a alternative solution and a plan that can be a Bridge class 1 that has load of plan traffic current 100%from total load before. With width of highways 7,00 m (double ways) then must uses 2 unit box girder shaped trapesium with extend of bridge 60 m with spesification

: tall of girder 2,80 m, width of bridge 2 x 8,50 m, quality of prestressed concrete box girder K-500 (fc=41,50 MPa), and quality of plat concrete/slab of floor bridge K-500 (fc = 41,50MPa).

2. Research Method

The steps in the analysis of the structure of the bridge on this final project is done with several stages, such as the purpose of calculating the width, and the span of the bridge onthe structure of the bridge. The steps of planning the bridge structure can be seen in figure.

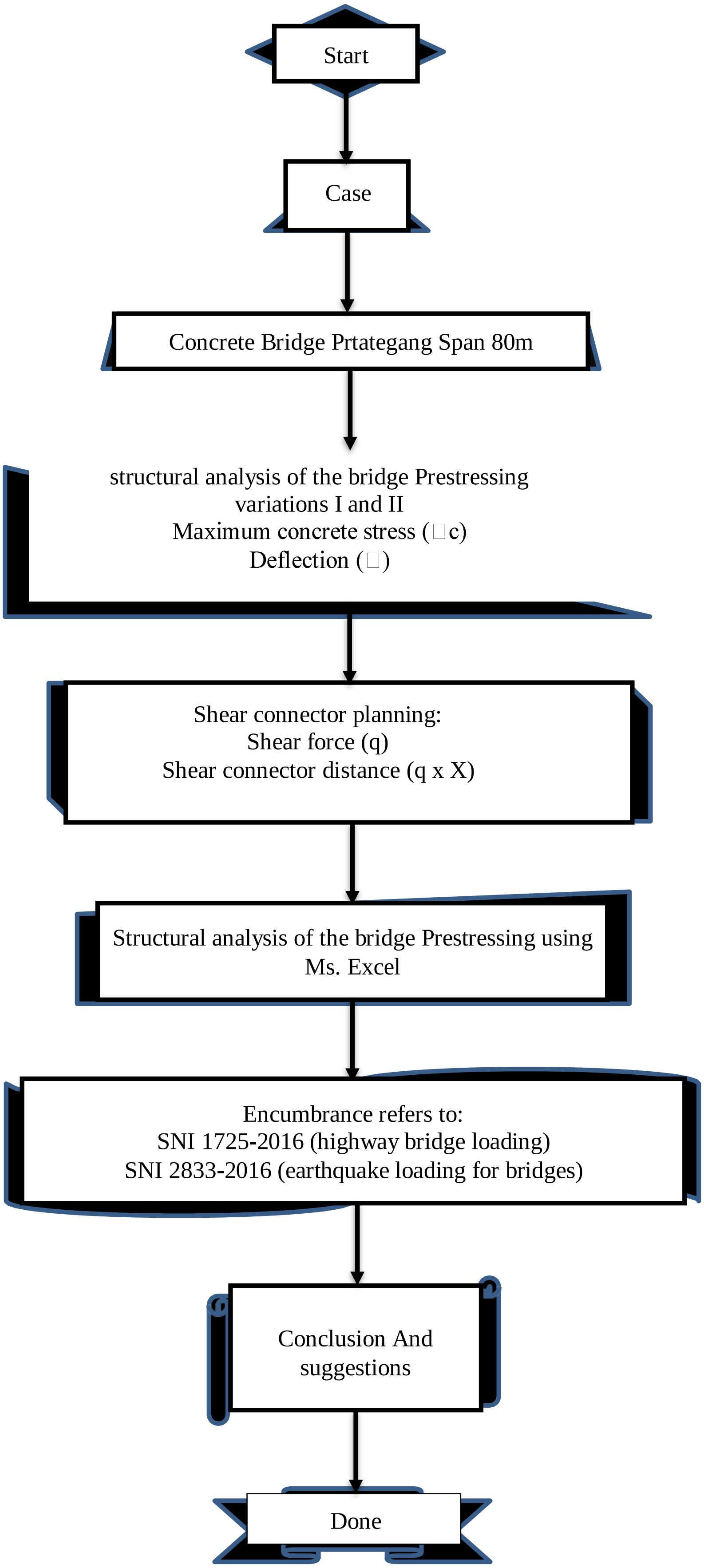

Figure 1. Research Methods

3. Result and Discussion

Table 1. Beam Moment Resume

Action / Load	Concrete Factor		Moment		Ultimate Moment	
	Ultimate		M	(kNm)	MU	(kNm)
A. Fixed action						
Own weight	K_{MS}	1,3	M_{MS}	24898,6	$K_{MS} * M_{MS}$	32368,18
Additional Dead Load	K_{MA}	2,0	M_{MA}	6683,2	$K_{MA} * M_{MA}$	13366,40
Shrink and crawl	K_{SR}	1,0	M_{SR}	3095,9	$K_{SR} * M_{SR}$	3095,86
B. Transient Action						
Column Load "D"	K_{TD}	2,0	M_{TD}	7094,4	$K_{TD} * M_{TD}$	14188,80
Brake Force	K_{TB}	2,0	M_{TB}	43,7	$K_{TB} * M_{TB}$	87,36
C. Environmental Action						
Influence of temperature	K_{ET}	1,2	M_{ET}	917,5	$K_{ET} * M_{ET}$	81857,13
Wind Load	K_{EW}	1,2	M_{EW}	65600,0	$K_{EW} * M_{EW}$	78720,00
Earthquake Load	K_{EQ}	1,0	M_{EQ}	81857,1	$K_{EQ} * M_{EQ}$	81857,13

The results of the above parameters get moments such as the table above.

4. Conclusion and Suggestions

From the analysis that has been done can be concluded that:

1. The prestressed bridge with a width of 1.20 m with a safe and optimal span of 80m has the following ultimate moments and moments: Fixed Action self weight moment is 24898.6 kNm and self weight ultimate moment is 32368.18 kNm, additional dead load moment is 6683.2 kNm and additional dead weight ultimate moment is 13366.40 kNm, shrinkage and creep moment is 3095.9 kNm and shrinkage and creep ultimate moment is 3095.86 kNm, prestressing moment is -15829.4 kNm and prestressing ultimate moment is -15829.42 knm, transient action “d” lane load moment is 7094.4 knm and “D” lane load ultimate moment is 14188.80 KNM, brake force moment is 43.7 knm and brake force ultimate moment is 87.36. Environmental action the moment of influence of temperature is 917.5 kNm and the ultimate moment of influence of temperature is 1101.03 kNm, the moment of wind load is 80960.0 and the ultimate moment of wind load is 97152.00 kNm, the moment of earthquake load is 81857.1 kNm and the ultimate moment of earthquake load is 81857.13kNm
2. Modeling analysis and pekrjaan building structure on prestressed concrete building bridges can mrnggunakan MsExcel, using a width of 1.20 m, and a length of 80 m mendaptkan final value of concrete moments prtegang and ultimate moment as well as the moment of his permission listed in Ms.Excel as mediation of prestressed concrete count.
3. the need for special details regarding bridge planning standards so that it has SNI in accordance with the circumstances and situation in Indonesia so that it does not need to use other country standards.
4. on the structure of prestressed concrete planning is still in need of a more thorough feasibility study and a complete reference.

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Study of the Stability in the High Building: Case Study in Burj Dubai, UAE

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Abstract

Many people today live in urban cities and it is increasing every year, because of their need to live and work and This paper summarizes the solutions and the importance of stability in high-rise buildings and their tolerance of earthquakes and winds, and all that is needed for the stability of the building and to solve the problems that occur in particular and in detail Highlighted are the key connections of the tallest and ultra-slender buildings that have been designed and built for the world's tallest buildings. He talks about high-rise towers in general and about Burj Dubai in particular, and there will be what the engineers said and suggested before, and I will conclude in a simple way.

Keywords :
High - Rise Buildings, Stability, Urban Cities, Wind Loads

1. Introduction

The first tall buildings were built in the 1880s in the United States, took up less space and became staffed with steel structures and outdoor glass enclosures, and in the 20th, century became an important feature in most cities. High-rise towers are considered one of the most important components of the country's economic strength and a sign that distinguishes it from other countries. Many countries are now seeking to make progress in building high-rise investment projects. Countries are advancing in planning and scrutinizing architectural aspects. Architectural progress in the country encourages technological progress to take advantage of the latest systems according to the company's standards. In the stability of these buildings due to the different loads in tall buildings than in low buildings because they are exposed to the wind, the increase in the height of the building increases the impact of increased winds loads, causing safety concerns for the structure envelope and the integrity of the structural system. Tall buildings shook along the wind. The vibrations may be large to cause anxiety and discomfort for the people present, or by strong earthquakes. Strong earthquakes are rare, but they may occur once a year and cause huge disasters, as happened in the Sumatra earthquake in 2004 and led to the death of more than 230 thousand people and many Fatalities as a result of building collapse, there is a high probability that tall buildings will be exposed to these vibrations that may occur and The most important factor in the design of tall buildings is the need to withstand external forces through the use of cross walls to provide a rigid structural frame with greater strength to withstand wind pressures. Assessment of building movement is a prerequisite, stability under wind load is important for structural structures. Safety and Architectural Safety Hazards can be reduced in tall buildings, especially in active areas with earthquakes or prevailing wind loads.

2. Literature Review

2.1. Historical Stages of the Construction of High-Rise Buildings

The construction of tall buildings passed through several historical stages and economic changes, and the first stage was the nucleus of building tall buildings or gigantic skyscrapers. The first, which consists of a block and a clapboard-like shape that results in a very repetitive pattern and buildings in this phase were constructed exclusively for commercial purposes and commercial activities required to be close to each other and to the city center and then the second architectural phase of tall buildings was more evident in New York buildings. This new architectural style is known as Art Deco. The third stage represents the entrance to high-rise building planning for marketing experts, resulting in tall buildings being a profitable symbol rather than a work of art. tall buildings began to have as many corner offices as possible. The fourth phase began in the 1980s and was an answer to the Cubism period by continuing the marketing trend in the skyscraper facade and maximizing the viewing angle by curves and retrogrades. Other major concerns around this style were creating amenities in the entrance foyer, avoiding the flat roof and declaring the building's small facade hence the fifth phase began in the 1990s. One of the most prominent features of this phase is the modification and improvement of building forms and energy efficiency. It deals with the passive phase in architecture, such as the effect of natural light and solar orientation, as well as the negative control of architecture and engineering, but energy efficiency can be achieved. With more comprehensive options such as improved insulation, use of building thermal mass and wind energy, energy could be the future of sustainability for tall buildings.

2.2. Needs for High-Rise Buildings for Stability

Most high-rise buildings contain steel frames, and concrete frames consist of columns (vertical support and horizontal support) and to provide a structural frame with greater rigidity, cross walls or shear walls are used to better withstand wind pressures When designing tall structures, basic requirements such as loads, resistance, stability and durability must be considered.

2.2.1. Loads

Vertical loads, i.e., dead and live, do not cause any design problems as they are mostly deterministic in nature. However, lateral forces from wind or earthquakes make the design unpleasant. These lateral forces can generate significant tensile stresses in the structure, cause unwanted vibrations, or cause excessive lateral tilts and deflections of the structure, and thus require unique and important considerations.

Advances in the design of multi-store structures have emphasized the importance of restricting lateral impact (displacement) as well as the activity of lateral loads. The provision of a shear wall, in contrast to conventional inflexible frames, reduces the side effect of the structure. A well- designed shear wall provides structural integrity and security for non-structural elements such as suspended ceilings, architectural walls of various types, and so on from seismic disturbances.

2.2.2. Resistance

One of the most important aspects of a tall structure is to withstand and stabilize the worst possible range of loads that may occur throughout the life of the structure, including the construction period. In addition, strains from differential activities such as creep, shrinkage or temperature must be included in the structural strength parameters.

2.2.3. Stability or Balance

High structures must have bases and foundations that ensure stability under the influence of a combination of vertical and lateral forces. Thus, foundations and structures must be designed and implemented so that the easy rule of net force flow is applied and passed through the middle third of the base, thus ensuring that the stresses under the foundations are compressive stresses and prevent the generation of extreme stresses. They cause the entire structure to tip over due to the movement of the rigid body around one edge of the base. Similarly, the resistance torque of the dead weight of the structure must be greater than the overturning torque of a suitable safety factor.

2.2.4. Durability

Structures built with the right materials and methods will be more robust and resilient than a building built without building codes and codes and using untrained labor even if it is a small or short building The longevity of reinforced concrete depends on the following factors in addition to the quality of materials and construction

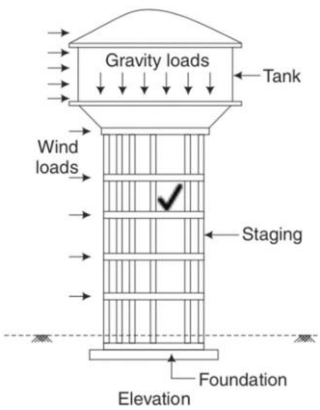

Figure 1. Group Loads on Structures

2.3. Needs for High-Rise Buildings for Stability

The force of the wind affects tall buildings of More than 6 floors or the shelters of mosques, high tanks and other high buildings because it adds an additional load (moment) on columns and foundations to increase the design stresses of structural and other elements and the depth of foundations under the soil. Therefore, wind and earthquake resistance must be taken into consideration structures when designing It is calculated from the following law: - F = A*P P= Ce*Cq*Qs*lw.

The impact of the wind in high towers is resisted by reducing the area exposed to the wind as we rise vertically, and this also gives greater stability to the tower, as the weight of the structure is greater at the base and first floors and gradually decreases as we go vertically upward. It is important to design tall buildings to resist wind and earthquake forces, especially in tall and slender buildings, because they are more sensitive and exposed to them. High altitudes from vibrations caused by wind and earthquakes, absorbing vibrations to reduce horizontal displacement of floors before they travel to upper floors. One of its applications is the tuned block damper, which consists of a block, spring, and damper attached to the facility and act as pendulums to reduce the dynamic response of the buildings and resist movement caused by wind or earthquake, in the event that an earthquake or wind causes the building to sway, the block moves in the opposite direction to restrict the movement of the building and the block absorbs energy kinetics to eventually dissipate through the dampers. This system is usually placed on the upper floors to control the lateral movement of the facility.

1 Based on Zeeshan et al., (2018) The most important factor in the design of tall buildings is the building's need to withstand lateral forces imposed by wind and earthquake. Cross walls or shear walls can be used to provide a structural framework with greater rigidity to withstand wind pressure. Square column behavior is better than rectangular column in terms of floor wear and base and roof shear offset. Shear walls are used to eliminate lateral load and soft flooring effects in shear walls, they are kept centrally and are not much affected by the behavior of structures.

2 Based on Sharma et al. (2017) Tall buildings are subject to dynamic horizontal loads such as winds and earthquakes. These horizontal forces cause significant stresses, displacements and vibrations due to the height and flexibility inherent in the building. Wind displacement and vibration become critical as altitude increases.

3 Based on Malik & Muhammed (2017) The demand for tall buildings is increasing day by day due to population growth and land shortage in urban areas. It is the multi-storey high-rise buildings that are affected by lateral forces due to their height to the extent that they play an important role in the structural design. Lateral stability is most important for tall buildings.

4 Based on Ding & Kareem (2020) The increasing growth of tall buildings in urban areas around the world places new demands on their performance in the wind. This includes choosing a building form that minimizes wind loads and a structural topology that transfers loads efficiently. Aerodynamic form design consisting of modifying the building's exterior has shown great promise in reducing wind loads and associated structural movements as reflected in the design of Taipei 101 and Burj Khalifa. In these buildings, angle adjustments for cross-section and taper are introduced along the height. An attractive alternative is the design of a building that can adapt its shape to the complex, changing wind environment of urban areas with clusters of high-rise buildings, that is, by implementing a dynamic facade.

5 Based on Hickey et al., (2022) High- rise buildings are subject to wind-induced movement which can lead to serviceability and habitability issues associated with occupant discomfort. Modular construction, in which volumetric units are assembled around a reinforced concrete core, can be particularly susceptible to wind-induced acceleration due to the long slender shape of the core and the units' small and uncertain contribution to global lateral stiffness and damping. There are two approaches to mitigating excessive vibrations; Increase the basic dimensions or add additional damping.

6 Based on Al-Kodmany (2018) As cities adapt to rapid population growth—adding 2.5 billion people by 2050—and struggle with expansionary expansion, politicians, planners, and architects have become increasingly interested in the vertical city model. The three pillars of sustainability are simply: social, economic and environmental. The identification of "unsustainable" aspects forms the basis for addressing them in future research and development of tall buildings.

7 Based on Avini et al,. (2019) The design of low to mid- rise buildings is based on a quasi-static analysis of wind loading. These measures do not fully address issues such as interference from other structures, wind direction, cross-wind response, and dynamic effects including acceleration, structural rigidity and damping that affect occupant comfort parameters.

8 Based on Nizamani et al., (2018) Wind is a randomly changing dynamic phenomenon consisting of many eddies of different sizes and characteristics of rotation along a general stream of air moving relative to the Earth. These eddies give off their winds, which creates fluctuations and results in complex flow characteristics. The wind vector at any point can be thought of as the sum of the mean wind vector and oscillation components. These components vary not only with altitude, but also with terrain. Prevailing winds exert pressure on structural surfaces.

9 Based on Jamaludden & Banerjee (2021) Many tall structures, tall towers, and buildings are now being built around the world, around the effect of wind load on tall buildings. Usually there are two buildings in the form of a symmetrical building. Due to some engineering or architectural requirements, the symmetry between the wings is not properly maintained, and the height of the building varies. This study

is about building displacement, span, base shear, and fundamental moment coefficients for a different shape of tall building. By varying the height of the building between 16 to 20 floors.

10 Based on Zaborova et al.,(2018) Through experiments with different construction materials. The experiments were conducted in a thermal chamber with a temperature of 0 to 20 °C (cyclical and non-cyclic) without thermal insulation materials with sensors installed throughout the structure to check the temperature. Then, overlapping graphs were drawn in order to compare the obtained results; It shows the heterogeneity of heat transfer in different parts of the structure within clay bricks (full-bodied), lightweight concrete and reinforced concrete. Finally, the results indicated that clay bricks are more thermally stable.

3. Research Methodology

The design of superstructures is subject to interaction with lateral winds and gravitational loads, and the choice of building form and structural system can greatly influence the wind and gravity behavior of a structure. In This paper outlines the design process and philosophy used in the design of the Burj Dubai's superlative buildings The concrete structure to keep the structure in its simple form, enhance buildability and reduce wind forces on the tower, Burj Dubai has been designed with the letter "Y". The skeletal system can be described as the core. Each wing assists, with its concrete core and high-performance perimeter columns, via a hexagonal central core, or hexagonal axis. The result is an extremely rigid torsion tower.

The structural system of the tower consists of a walkway, hammerhead-reinforced concrete walls and a central hex, and the reinforced concrete core walls provide hexagonal resistance to resist torsion of the structure, similar to a closed tube or axle. To renitence wind shears and moments for piles in the mechanical floors of the surrounding columns allow interference to jointly resist the lateral load of the structure, and all vertical concrete is used as support for lateral and gravity loads, and for the strength of concrete in the wall ranges from C80 to C60 cu. And local aggregates were used to design the concrete mixture, and fly ash and Portland cement were used. the reinforced concrete structure and building code requirements for structural concrete are designed. According to the requirements of ACI 318-02 American Concrete Institute The upper side of the tower consists of a structural steel tower and uses an inclined side system. Also in accordance with the requirements of the American Institute of Structural Steel AISC, the structural steel tower is designed to specification for design factor of resistance, wind, earthquake, pressure and resistance of structural steel buildings and resistance to loads (1999). Also, the exposed outer steel surface is protected by a flame-applied aluminum finish. The foundations of this tower consist of a raft supported by piles. The thickness of the solid reinforced concrete raft is 3.7 m. then the raft was cast from the SCC using 12500 m3 of C50 cubic force. in four (4) separate estuaries the raft is built (central core and three wings).

A 3D model of reinforced concrete walls, piles, slabs and mats and conical structural steel was analyzed and this structure consisted of hinged beams. Dubai system as a seismic zone UBC97 zone 2a with seismic zone factor Z = 0.15 and soil shape was determined by Dubai Municipality to protect the foundation of the tower, cathodic and other protection was used for reinforcements. The buoy is supported by 194 piles cast in place, approximately 43 meters long and 1.5 meters in diameter, with a capacity of 3,000 tons each. The tonnage of these piles has been tested up to 6000 tons. Then C60 concrete (cubic strength) was laid using a polymer slurry by Trim method. The friction piles are supported in naturally occurring calcite formations / agglomerated calcite formations. The resulting motions and wind forces at higher levels become dominant factors in structural design. An extensive program with proportional parametric layers for wind tunnel tests and other wind tunnel studies was conducted, 2.4m x 1.9m in RWDI and 4.9m x 2.4m in Ontario, Guelph. To develop aerodynamic solutions aimed at reducing wind speeds in these studies, studies were used Pedestrian winds at scale greater than 1:250 and 1:500 scale models. High-frequency power balancing technology was also used, and wind tunnel tests were conducted early in the design. The dynamic characteristics of the tower were then combined in order to calculate the dynamic response of the tower with wind tunnel data and the overall effective wind force distributions on a large scale.

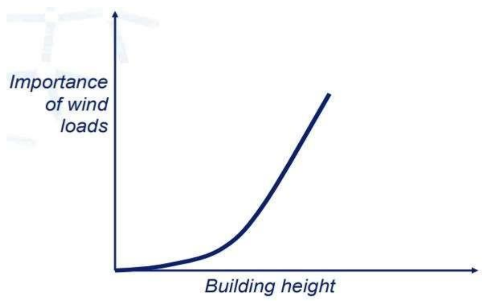

Figure 2. A curve showing the ascending relationship between the importance of increased wind force and the height of buildings.

The design used a technique that utilized in situ test data and pile installation data to determine the amount of construction control and performance monitoring of the supported structure during and after construction. ϕ_g may range from 0.4 for cautious designs involving little or no pile testing and unknown ground conditions to 0.8 for circumstances where substantial testing is done and the ground conditions and design parameters have been properly examined.

Table 1. Design test illustrative table

Situation	Purpose	Agent Applied to Geotechnical Force Parameters	Load Situation	Commentary
1	Structural design capacity	1.0	ULS	The maximum pile axial load and pile bending moment are calculated using unfactored geotechnical strength parameters and short-term pile modulus.
2	Geotechnical design ability	ϕg	ULS	The ϕg geotechnical reduction factor is used to strength measurements to determine the pile group's overall stability.
3	Serviceability	1.0	SLS	To evaluate pile head deflections and rotations, unfactored geotechnical strength parameters are used.

4. Discussion and Results

4.1. Foundation Design Analysis

A review of the hull foundations was conducted, and it was decided that the stacked foundations would be acceptable to both the tower and the platform. Final unit shaft resistance f_s = 0.25 (q_u) 0.5, 24 where f_s = uniaxial compressive strength in MN/m2 and q_u = final unit shaft resistance in kPa. The pontoon of this tower was built at -7.55 m DMD using tower piles 1.5 m in diameter and 47.45 m in length. The thickness of the raft was 3.7 meters. Loading, consisting of four wind loading scenarios and three seismic loading scenarios, the response of the more stable soil layer reduced the tower's stability to dead load, live load and wind load by 28%, from 85 to 61 mm. The load difference above or below the pile was looked at in order to better understand how cyclic loading affects the pile. It has happened sometimes to have dead load and live load. It was determined that the maximum load contrast of more than 10 MN was not sufficient.

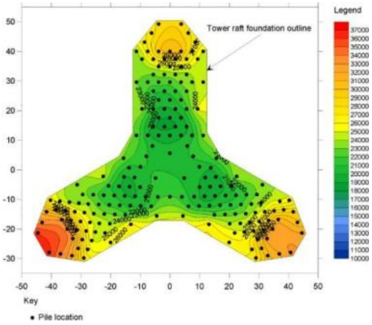

Figure 3. Maximum Axial Load Contours

The tower piles had a minimum center-to-center spacing 2.5 times greater than the diameter of the piles. Assuming that the foundation behaved as a mass of piles and soil or rock, a check was made to ensure that the tower's foundation was both vertically and laterally stable. For vertical block movement excluding base resistance, a safety factor of just under 2 was evaluated, while a safety factor of over 2 was rated for lateral mass movement excluding passive resistance. A safety factor of approximately 5 against block inversion was obtained.

4.2. Independent Review Analysis

Tabel 2 provides a summary of the geotechnical model that was used in the validation analyses.

Table 2. Summary of the Geotechnical Model

Layer Number	Description	RL range DM	Undrained Modulus Eu (MPa)	Drained modulus E0 (MPa)	Definitive Skin Friction (kPa)	Definitive End Bearing (MPa)
1a	Medium-density silt sand	+2.5 to +1.0	30	25	-	-
1b	Loose-v. loose silty sand	+1.0 to -1.2	12.5	10	-	-
2	Weak calcarenite. Weak-mod.	-1.2 to -7.3	400	325	400	4.0
3	V. weak calc. sandstone	-7.3 to -24	190	150	300	3.0
4	V. weak-weak sandstone/calc. sandstone	-24 to -28.5	220	175	360	3.6
5a 5b	V. weak–weak–mod. Weak calcisiltite/conglomerate	-28.5 to -50	250	200	250	2.5
	V. weak–weak–mod. weak calcisiltite/conglomerate	-50 to -70	275	225	275	2.75
6	Calcareous siltstone	-70 and below	500	400	375	3.75

The following methods were used to investigate the potential effects of cyclic loading:

Periodic three-axis laboratory tests, direct periodic shear tests, periodic tests of constant normal stiffness (CNS), and periodic theoretical analysis performed by an independent auditor are all acceptable methods of verification. Direct shear tests revealed a decrease in residual shear strength, although performed using high stress levels that were not indicative of potential field conditions. Periodic triaxial tests revealed some potential for stiffness deterioration and accumulation of excessive pore pressure. As long as the cyclic shear stress remains within the expected range, CNS tests have shown that there is no significant risk of cyclic deterioration of cutaneous friction.

4.3. Test of Outrigger Loads

The Burj Khalifa project implemented two static load testing programs namely static load tests on seven pilot piles prior to foundation construction and also during the foundation construction phase, static load tests were performed on eight piles (about 1 percent of the total number of built piles). In addition, the dynamic pile test was performed on 10 tower work piles and 31 platform work piles, or about 5% of the total number of work piles.

Table 3 provides a summary of information about the piles tested as part of this program. The main objectives of the tests were to validate the design hypotheses and evaluate the general load leveling behavior of piles of expected length below the tower. Since each test pile was unique, several factors could be examined, as follows:

The effects of pile lengthening are as follows: shaft injection effects, shaft diameter reduction effects, (tensile) loading effects, side loading effects, and cyclic loading effects. Instead of the more common bentonite drilling fluid, piles are built using polymer drilling fluid. The use of the polymer appears to have produced piles with above-average performance. Each pile has stress gauges installed along its length, allowing a comprehensive analysis of load transfer along the shaft as well as an assessment of the packed skin friction distribution with depth along the shaft. The reaction system used in axial load tests consists of four or six contiguous reaction stacks, depending on which stack is being tested. These reaction piles can interact with the test pile through the soil and affect the results of the substrate load tests.

Table 3. Pile load test summary—preliminary pile testing

Pile no.	Pile diameter (m)	Pile length (m)	Side grouted?	Test type
TP1	1.5	45.15	No	Compression
TP2	1.5	55.15	No	Compression
TP3	1.5	35.15	Yes	Compression
TP4	0.9	47.10	No	Compression (cyclic)
TP5	0.9	47.05	Yes	Compression
TP6	0.9	36.51	No	Tension
TP7A	0.9	37.51	No	Lateral

None of the six axial load tests appear to have reached maximum axial capacity, at least not in terms of geotechnical resistance. Test piles with a diameter of 0.9 m (TP4 and TP6), 4.5 m (TP5) and 1.5 m (TP1, TP2, and TP3) were loaded to 3.5 times the working load, four times the working load, and twice the working load, respectively. None of the other pillars, besides TP5, showed a stark indication of impending geotechnical failure. At maximum load, TP5 showed a strong increase in failures; However, this has been attributed to the structural failure of the stack itself. An important finding from a design perspective was that the geotechnical failure safety factor at the workload appears to be greater than 3, providing a comfortable margin of safety against failure, especially since the raft would also provide additional resistance to supplement that of the belt.

Behavior when load leveling: Substrate settlements measured at working load and maximum test load are listed in Table 4, along with relevant values for pile head stiffness (load/levelling). Subsequent results are made:

The stiffness was higher for piles with larger diameters as expected. The stiffness of the shaft- immunized substrates (TP3 and TP5) was higher than that of similar non-grooved substrates. The measured stiffness values were relatively large and well above those expected.

Table 4. Summary of Axial Loading Findings From Pie Load Test

Stack number	operating load (MN)	Max. load (MN)	Settlement at W. load (mm)	Settlement at max. load (mm)	Stiffness at W. load (MN/m)	Stiffness at max. load (MN/m)
TP1	30.13	60.26	7.89	21.26	3819	2834
TP2	30.13	60.26	5.55	16.85	5429	3576
TP3	30.13	60.26	5.78	20.24	5213	2977
TP4	10.1	35.07	4.47	26.62	2260	1317
TP5	10.1	40.16	3.64	27.45	2775	1463
TP6	-1.0	-3.5	-0.65	-4.88	1536	717

In light of the alleged low boulder grade at the Burj Dubai site, the pillar head stiffness values for Burj Dubai piles were expected to be slightly lower than those of Emirates Towers based on the experience gained at the adjacent Emirates Project site. The enhanced performance of the piles in the current project can be attributed, at least in part, to the use of polymer drilling fluid instead of bentonite during the construction process. It was also conceivable that the interaction effects of the reaction piles played a role in at least some of the higher stiffness values. Response piles suffer from tension and subsequent rise when a compressive load is applied to the test pile, which tends to reduce the tendency of the test pile to settle. Therefore, the apparent high pile stiffness may not accurately reflect the actual pile stiffness under the building.

4.4. Cyclic Loading Effects

After achieving the working load in each of the axial load tests, the heap experienced a very limited number of load cycles. The test results that were extrapolated from the pregnancy stabilization data are summarized in Table 5. Both adjustments occurred at the maximum load of the spin cycle, and the post-cycle settlement was connected to the first-cycle settlement. It is clear that under the influence of cyclic loading, settlements accumulated, despite the relatively high levels of medium pressure and cyclic pressure of the aggregate, this accumulation was very slight (in all cases, the maximum load reached was 1.5 times the working load). These results were consistent with estimates made during the design, which indicated that the effects of cyclic loading would likely not have a significant impact on this structure.

Table 5. Displacement accumulation for Cyclic Loading Summarized

Stack number	Mean load/Pw	Cyclic load/Pw	No. of cycles (N)	SN/S1
TP1	1.0	±0.5	6	1.12
TP2	1.0	±0.5	6	1.25
TP3	1.0	±0.5	6	1.25
TP4	1.25	±0.25	9	1.25
TP5	1.25	±0.25	9	1.3
TP6	1.0	±0.5	6	1.1

Positive and reassuring information on the capacity and stiffness of the piles was provided by both the preliminary test piling program and the tests on the work piles. The measured pile head stiffness values far exceeded the expected values. Although the increased apparent pile head stiffnesses may have been caused by interactions between the test and reaction piles, the heaps nevertheless went above and above expectations. Although none of the tests fully mobilized the potential geotechnical resistance, it appeared that the piles' capacity was greater than the projected values. Up to a maximum test load of 1.5 times working load, the works piles behaved even better than the earlier trial piles and showed nearly linear load-settlement behavior. Given the excellent performance of the unrouted piles, it was determined that shaft grouting would not be necessary for this project. Shaft grouting appeared to have improved the load-settlement response of the piles. Although it was acknowledged that the overall settlement behavior (and possibly the overall load capacity) would depend not only on the individual pile characteristics but also on the characteristics of the ground within the structure's zone of influence, it was inferred from the pile load test data that the design estimates of capacity and settlement may be conservative.

4.5. Execution of Settlement During Construction

After the pouring of concrete was completed, the settlement of the tower raft was observed. Stress rosettes were placed at the top and bottom of the raft to measure the stress conditions inside. In order to evaluate the load distribution between and below the pile, the pressure of five piles was measured in addition to the three pressure cells which were placed at the base of the raft. Measured adjustments will be found to be less than those expected during the design process. The piling system would still have to undergo some dead and direct loading, and it must be emphasized that the observed values do not take into account the effects of rafting, cladding, and direct loading, which would account for more than 20% of the total volume. Final settlement was estimated to be about 55-60mm, which is comfortably lower than the expected final settlement by about 70-75mm, and extrapolated to full dead load plus live load. The total maximum expected settlement may drop to approximately 52 mm. The measured settlement parameters are shown in Figure 4. The general distribution matches what the diversified analysis predicted. The calculated shortening of the structure after 30 years was expected to be about 300 mm, much more than the foundation settlements, to put the foundation settlements into perspective.

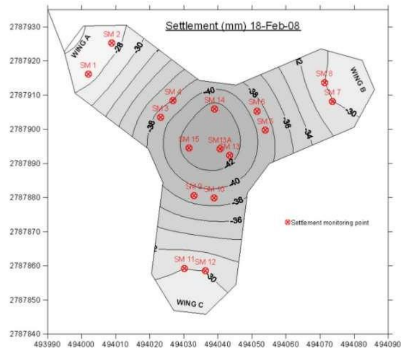

Figure 4. Measured settlement contours as of February 2008

5. Summary and Conclusion

Many people today live in urban cities and they are increasing every year due to their need to live and work. High-rise towers are considered one of the most important components of the country's economic strength and a sign that distinguishes it from other countries. Attention and focus must be given to the stability of these buildings due to the different loads in tall buildings compared to low buildings because they are exposed to wind, and an increase in building height increases the impact of increased winds and increased wind loads, causing safety concerns for the structure envelope and the integrity of the structural system. The possibility of high-rise buildings being exposed to these vibrations that may occur, and the most important factor in the design of tall buildings is the need to withstand external forces through the use of cross walls to provide a solid structural frame with greater strength to withstand wind stresses. Assessment of building movement is a prerequisite, and stability under wind load is important for structural structures.

Some methods have been used to investigate potential effects of cyclic loading and they are triaxial cyclic laboratory tests, direct cyclic shear tests, periodic tests of constant normal stiffness (CNS), and cyclic theoretical analysis performed by an independent auditor are all acceptable methods of verification. Direct shear tests revealed a decrease in residual shear strength. Periodic triaxial tests revealed some potential for stiffness deterioration and excessive pore pressure build- up. As long as the cyclic shear stress remains within the expected range. It is clear that under the influence of periodic loading, settlements accumulated and this accumulation was very slight. These results were consistent with estimates made during the design, which indicated that the effects of cyclic loading likely did not have a significant impact on this structure.

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The Effect of Adding Fly Ash and Glass Powder on the Compression Strength of Concrete

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Abstract

The purpose of this study was to determine the effect of the use of fly ash waste and glass powder on the compressive strength of concrete. 17.5% at the age of 3 days, 7 days, 14 days, and 28 days. for the specimens used measuring with a diameter of 10 cm and a height of 20 cm as many as 36 test samples by making variations of the day as many as 3 samples of the test object, After testing the compressive strength of concrete, the compressive strength of the characteristic concrete with the addition of 7% fly ash and glass powder 5%, the maximum compressive strength at the age of 7 days was 23.35 Mpa, the age of 14 days was 28.93 Mpa, and the age of 28 days was 33.85 Mpa. These results exceed the value of the compressive strength of normal concrete characteristics and indicate that fly ash and glass powder increase the compressive strength of concrete. There is a strong influence from the addition of fly ash and glass powder with variations in the addition and certain age of concrete

Keywords:
Compressive Strength, Concrete, Fly Ash Waste, Glass, Alternative.

1. Preliminary

1.1. Background

In the current era, the increasing number of glass industrial plants also results in a large amount of glass waste from production residues. Some glass waste from production waste will usually be made of new glass and other waste is disposed of directly on the ground or in the river without being used properly this can cause environmental pollution. Many studies have been carried out to utilize the glass waste into something more useful, namely as a mixture of concrete mixtures, where the glass can be processed into glass powder. The use of glass powder waste as a concrete mix is expected to reduce unused glass waste. Concrete is a mixture of portland cement or other hydraulic cement, coarse aggregate, fine aggregate, and water with or without additives that form a solid mass (SNI-03-2847,

Glass is an amorphous material made by dry silica and basic oxides. The hardness of glass provides abrasion resistance to concrete. Glass was chosen as an additional material for the concrete mixture, because it can increase the compressive strength of concrete.

Glass powder or fritz is glass flakes that are crushed and can be made into ceramic mixtures in ceramic factories. This glass powder is a fine grain measuring 0.075mm – 0.12mm, non-porous and pozzolanic. Glass powder can be expected to increase the compressive strength of concrete because the grains are very small and are able to fill the pores in the concrete.

Fly Ash(fly ash) is one of the residues produced in combustion and consists of fine particles from the combustion of coal. Fineness of fly ash

This has the potential to cause air pollution. In addition, fly ash handlers are currently limited on stockpiling in vacant land the use of fly ash as a concrete-forming material has a positive impact from an environtmental point of view.

As a concrete mixture, it shows that the value of the compressive strength of concrete at the age of 28 days with fly ash content of 7%, 9& and variations in glass powder content of 7%, 5% obtained the optimum compressive strength value at a variation of 7% glass powder content.

In this study, the author tried to add a mixture of concrete with glass powder 7%, 5% and fly ash variation levels of 7%, 9% to find out the maximum limit of the percentage of fly ash that is good for the compressive strength of concrete.

Based on this background triggered a research entitled "Effect of Addition of Fly Ash and Glass Powder on Compressive Strength of Concrete" utilizing fly ash and glass waste materials to reduce the use of cement as the main material for making concrete against compressive strength.

1.2. Formulation of the problem

Based on the background above, the formulation in this study is as follows:

1. How does the addition of fly ash and glass powder affect the compressive strength of normal concrete?
2. What is the optimum content of fly ash and glass powder added to achieve the design compressive strength?

3. On how many days variation of concrete with fly ash and glass powder variations can have the best results in how long?

1.3. Research Aims and Objectives

The aims and objectives of this research are as follows:

1. Knowing the effect of adding fly ash and glass powder to the compressive strength of normal concrete?
2. Knowing what is the optimum level of fly ash added to achieve the design compressive strength?
3. Knowing the compressive strength of concrete from the 14th, 21st, and 28th days

1.4. Restrictions and Scope of the Problem

Limitations of the problem in this study need to be made so that this research can be controlled and studied properly. Limitations of the problem in this study are:

1. The use of fly ash and glass powder as additives in the manufacture of concrete.
2. The water used is fresh water available at the Civil Engineering Laboratory, Mercu Buana University Campus D Kranggan.
3. The cement used is type 1 portland cement.
4. The fine aggregate used is sand.
5. Coarse aggregate used is crushed stone (split) with a maximum size of 25 mm.
6. Mix design using the calculation of SNI 03 28 24 2000 with a design compressive strength of 30 MPa and a w/c ratio of 0.33.
7. The tests carried out include; gradation of aggregate grains, specific gravity of aggregates, water content of aggregates, water absorption of aggregates, slump test of concrete, weight of test samples, and compressive strength tests of concrete using a compression machine.
8. Variations in the use of fly ash and glass powder as additives were 7%, 9% and 7% and 5% glass powder content.
9. The sample of the test object used was a cylindrical test object measuring 15 cm x 10 cm as many as 48 pieces.
10. The age of the specimens to be tested for the compressive strength of concrete is 14 days, 21 days and 28 days.

2. Literature Review

2.1. Definition of Concrete

Concrete (concerete) is a mixture of Portland cement or other hydraulic cement, fine aggregate, coarse aggregate, and water, with or without admixture. Along with increasing age, the concrete will harden and will reach the design strength (f'c) at the age of 28 days.

Concrete is formed by mixing rock material which is bound with an adhesive in the form of cement. The rock materials used to make concrete are generally divided into two types, namely, coarse aggregate (gravel/pecas/split) and fine aggregate (sand). Fine aggregate and coarse aggregate are referred to as mixed composite materials and are the main components of concrete. In general, the use of aggregate in concrete mixes reaches 70%-75% of all concrete (Aji & Purwono, 2010).

2.2. Concrete Forming Material

2.2.1. Portland Cement

Portland cement is the most common type of cement used generally throughout the world as a base material for non-specialized concrete, mortar, plaster and mortar. It was developed from other types of hydraulic lime in Great Britain in the mid-19th century, and is usually derived from limestone. This cement is a fine powder produced by heating limestone and clay minerals in a kiln to form clinker, grinding the clinker, and adding small amounts of other materials. Several types of Portland cement are available, the most common being called ordinary Portland cement OPC , is gray, but white Portland cement is also available.Portland cement as hydraulic cement cement which not only hardens by reacting with water but also forms a water-resistant product produced by crushing clinker consisting essentially of hydraulic calcium silicates, usually containing one or more forms of calcium sulfate as an inter-soil addition. Portland cement is hydraulic material consisting of at least two-thirds of the mass of calcium silicate, 3 CaOSiO2, and 2 CaOSiO2, the remainder consisting of an aluminum and iron-containing clinker phase and other compounds.the remainder consists of an aluminum- and iron-containing clinker phase and other compounds.the remainder consists of an aluminum- and iron-containing clinker phase and other compounds.

1. Type I (Ordinary Portland Cement)
2. Type II (Modified Portland Cement)
3. Type III (Rapid – Hardening Portland Cement)
4. Type IV (Low – Heat Portland Cement)
5. Type V (Sulphate – Resisting Portland Cement)

2.2.2. Aggregate

Aggregate is a material used as a filler for concrete. Aggregates are formed from volcanic eruptions, deposits, rock fragments or are produced unnaturally such as broken bricks, burning slag, and other synthetic materials. Aggregate is

main constituent of concrete to reduce shrinkage and lower production costs. Almost 70%-80% of the volume of concrete is aggregate (Duggal, 2008).

Aggregate is like a skeleton on concrete. The level of viscosity and the value of cohesion in freshly made concrete is influenced by the amount, type, surface texture, and size gradation of the aggregate used. In hardened concrete, aggregate greatly affects the value of stiffness, unit weight, strength, thermal properties, bonding, and wear resistance of concrete (Li, 2011).

2.2.3. Coarse Aggregate

Coarse aggregate is aggregate that does not pass through the No. 4 (4.75 mm) sieve. In general, the size of the coarse aggregate ranges from 5 mm to 150 mm. In normal concrete for structural elements such as beams and columns, the maximum size of coarse aggregate is 25 mm. In large concrete buildings such as dams and deep foundations, the coarse aggregate used can reach 150 mm (Li, 2011). Types of coarse aggregate are generally natural crushed stone, natural gravel, artificial coarse aggregate such as slag or shale, and aggregates for nuclear protection and heavy weights such as crushed steel, barite, magnetite andlimonite (Nawy, 1998).

2.2.4. Fine Aggregate

Fine aggregate is the aggregate that passes the No. 4 (4.75 mm) sieve and is retained on the No. 200 (75 m) sieve. River sand is the most commonly used type of fine aggregate. (zongli) Generally sand has a minimum size of 0.07 mm (0.003 in.). Materials measuring 0.06 mm (0.002 in.) to 0.02 mm (0.0008 in.) are classified as silt, and other smaller particles are classified as clay (Neville & Brooks, 2010).

2.2.5. Water

Water is a vital component in the manufacture of concrete. The amount of water used in the concrete manufacturing process greatly affects the resulting concrete. If the amount of water is too much, the cement will be lifted up with the water through capillary action and form a thin layer on the top surface of the concrete which is called laitance and weakens the bond in the concrete. In addition, too much water will cause the concrete to become hollow and porous when the water has been removed from the concrete. Meanwhile, when the amount of water is too little it will reduce workability and make fresh concrete difficult to use and consequently make the mixture of constituent materials spread. uneven concrete so that it reduces the strength of the concrete (Duggal, 2008).

2.2.6. Fly Ash

In the process of burning coal, two residual materials are produced. One material that comes out of the chimney of the furnace in the form of very fine dust is called fly ash. While the other material in the form of coarse dust at the bottom of the furnace is called bottom ash. has a high cement content and has pozzolanic properties. According to Neville & Brooks, (2010) pozzolanic properties are the properties of materials containing silica and alumina compounds. Fly ash content according to Santoso et al., (2004) contains Silica $SiO2$, Iron Oxide $Fe2O3$, Aluminum Oxide $Al2O3$, Potassium Oxide CaO, Magnesium Oxide MgO, and Sulfate $SO4$.

2.2.7. Glass Powder

Glass is one of the products of the chemical industry that is most familiar to our daily lives. From a physics point of view, glass is a very cold liquid. This occurs due to a very fast cooling process, so that the silica particles do not have time to arrange themselves in an orderly manner. From a chemical point of view, glass is a combination of various inorganic non-volatile oxides, which are produced from the decomposition and smelting of alkaline and alkaline earth compounds, sand and various other constituents.The characteristic properties of glass are influenced by the uniqueness of silica SiO2 and the process of its formation.Glass is a material made by dry silica with a basic oxide.The roughness of glass gives concrete a resistance to abrasion that can only be achieved by a few natural aggregate stones. Glass has distinctive properties compared to other ceramic groups. The reactions that occur in the manufacture of glass are summarized in equation 2-1 dian, 2011 Na2CO3 a.SiO2 Na2O.a.SiO2 CO2 CaCO3 b.SiO2 CaO.b.

2.2.8. Concrete slump test

The concrete slump test measures the consistency of fresh concrete before use. The test is used to test the workability of fresh concrete and the ease with which fresh concrete can flow (Gambhir, 2004). The slump test of concrete is carried out by decreasing the height at the center of the top surface of the concrete which is measured immediately after the slump test mold is lifted. Concrete with a slump value of 15 mm may not be plastic enough and concrete having a slump of 230 mm may not be sufficiently cohesive for this test.

2.2.9. Compressive Strength of Concrete

The compressive strength of concrete is the magnitude of the load per unit area, which causes the concrete test object to crumble when it is loaded with a certain compressive force generated by the press machine. The compressive strength of concrete is the most important property in the quality of concrete compared to other properties. The compressive strength of concrete is determined by the setting of the ratio of cement, coarse and fine aggregate, water. The ratio of water to cement, the higher the compressive strength. A certain amount of water is required to provide a chemical action in hardening concrete, excess water increases workability but decreases strength (Wang & Salmon, 1990).

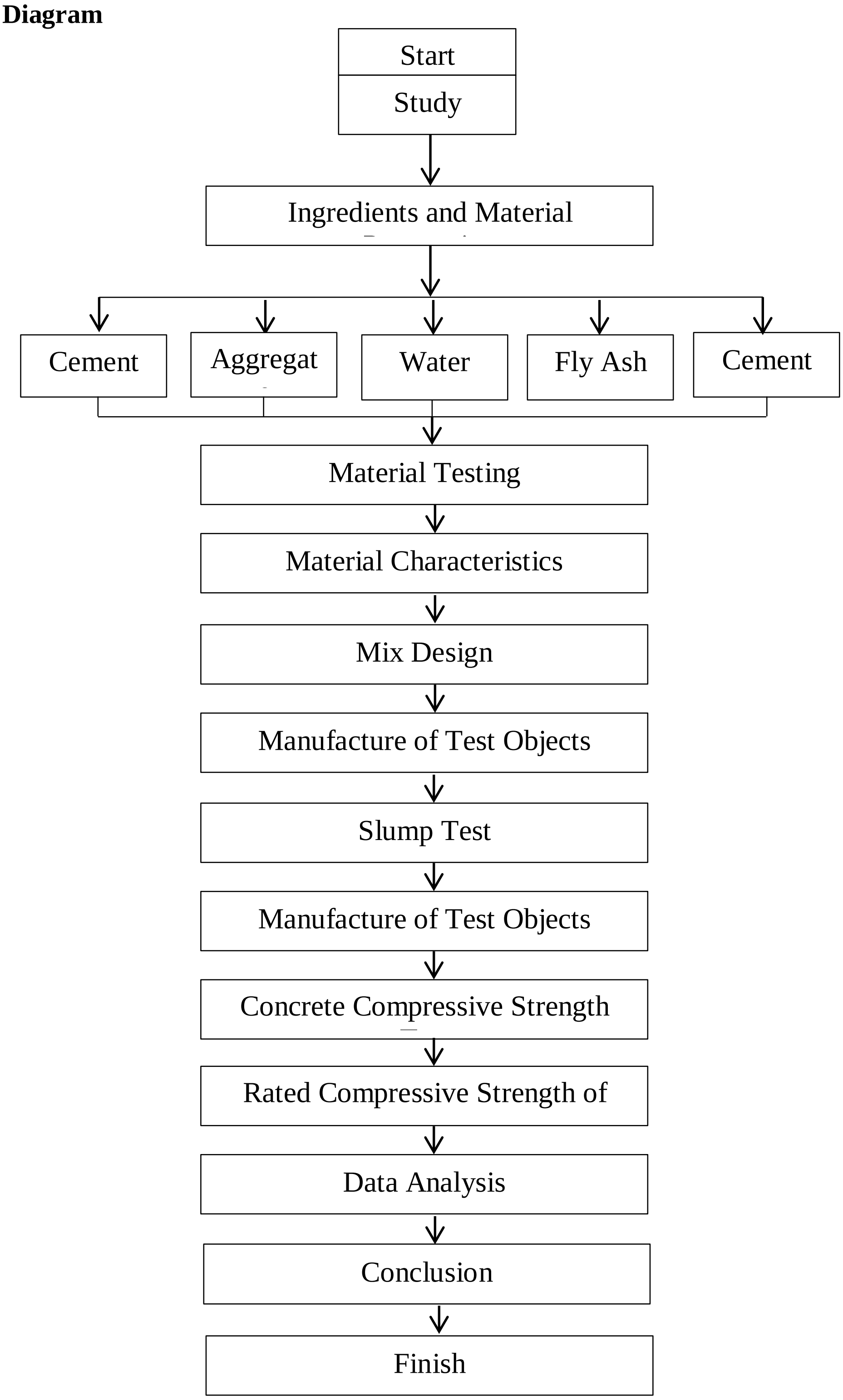
3. Research Methods

3.1. General Description

The research method used is the experimental method. The experimental method is a method to determine whether the cause of the independent variable affects the effect of the dependent variable. Hermawan, (2006) the independent variable to be used is the variation in the percentage of fine aggregate substitution using sawdust and pretreatment.

In this study, researchers will use a 10x20 cm cylinder. Researchers used Fly ash cement with a concentration of 5%, 9% and variations in the content of glass powder 7%, 5% researchers conducted a concrete compressive test on days 7, 14, 28, after the treatment process.

3.2. Flow Diagram



Flowchart 1. Flow Diagram

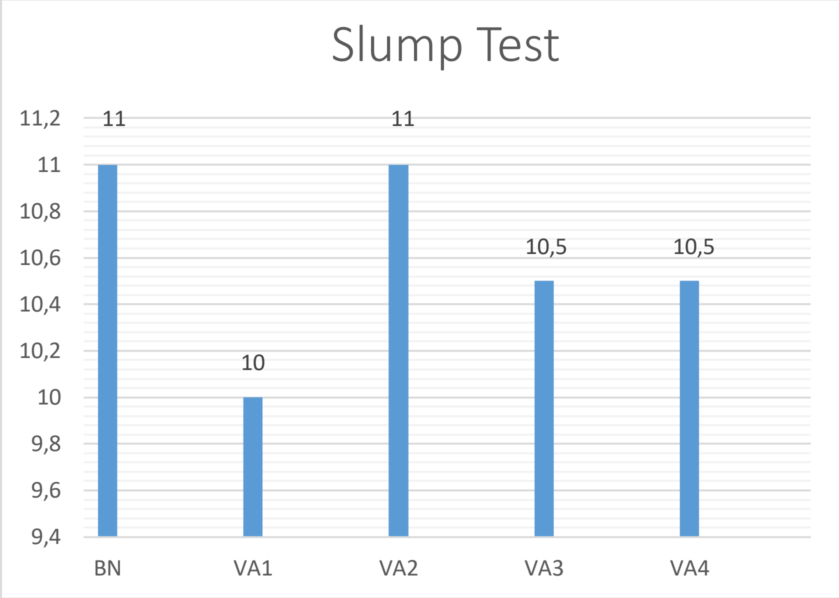
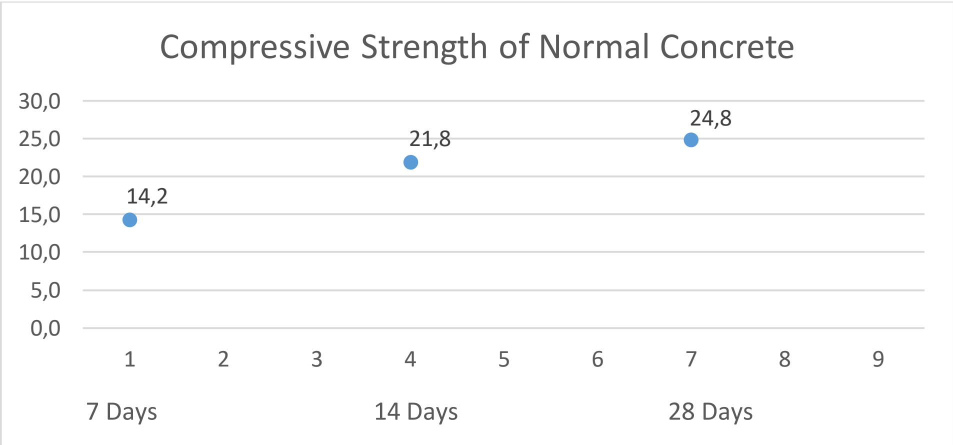


Figure 1. Slump Test Diagram

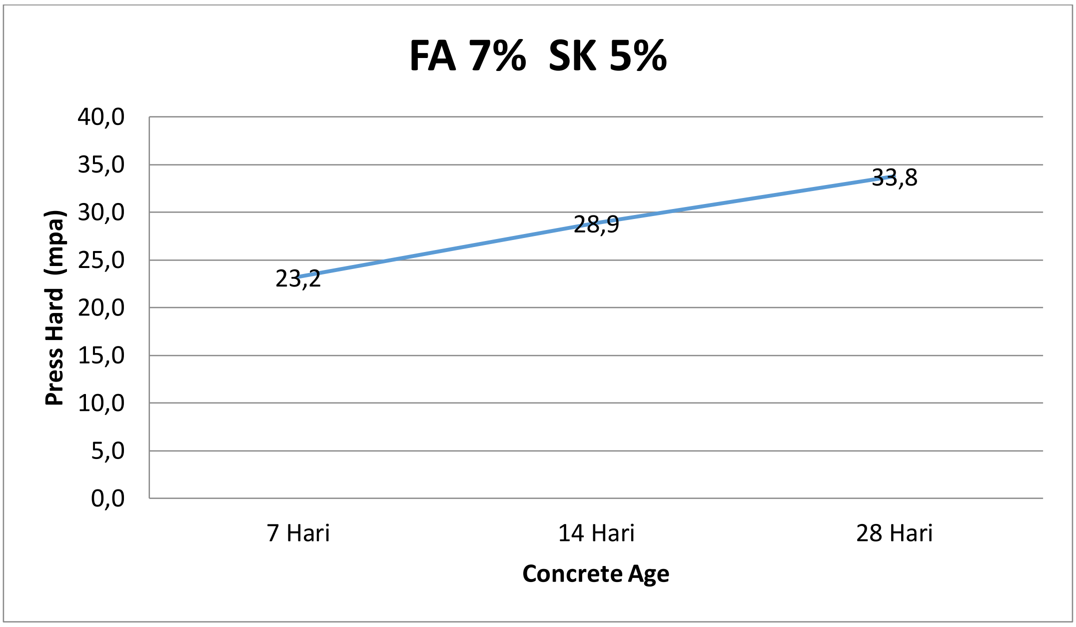
From the slump data, it is obtained when viewed from the overall test object BN, VA1, VA2, VA3, VA4, namely 11, 10.5, 9, and 11 have met the slump standard. So it can be concluded that the higher the percentage of addition of fly ash and glass powder, the smaller the slump obtained, this is due to the nature of fly ash and glass powder that easily absorbs water in the concrete mix so that the concrete mixture is getting thicker.



Source: Research Data

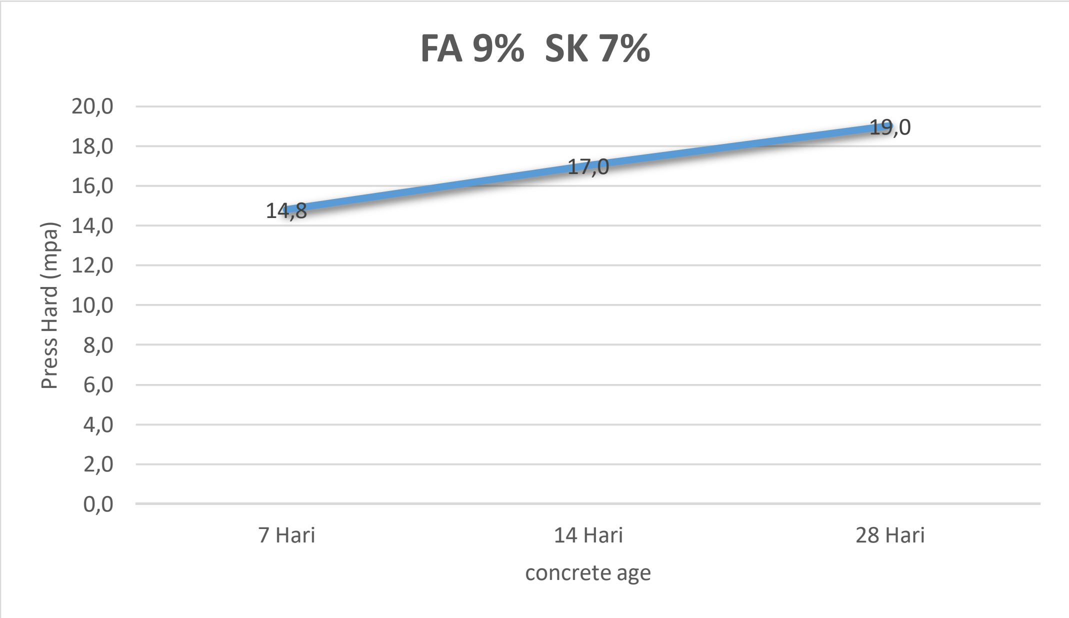
Figure 2. Graph of Normal Concrete Test Results

From the table above, it is known that the normal compressive strength of concrete on day 7 is 14.20 MPa on day 14, which is 21.80 MPa on day 28, which is 24.80 MPa. This can also be seen in the graph that the increase continues to occur until the age of 28 days.



Source: Research Data

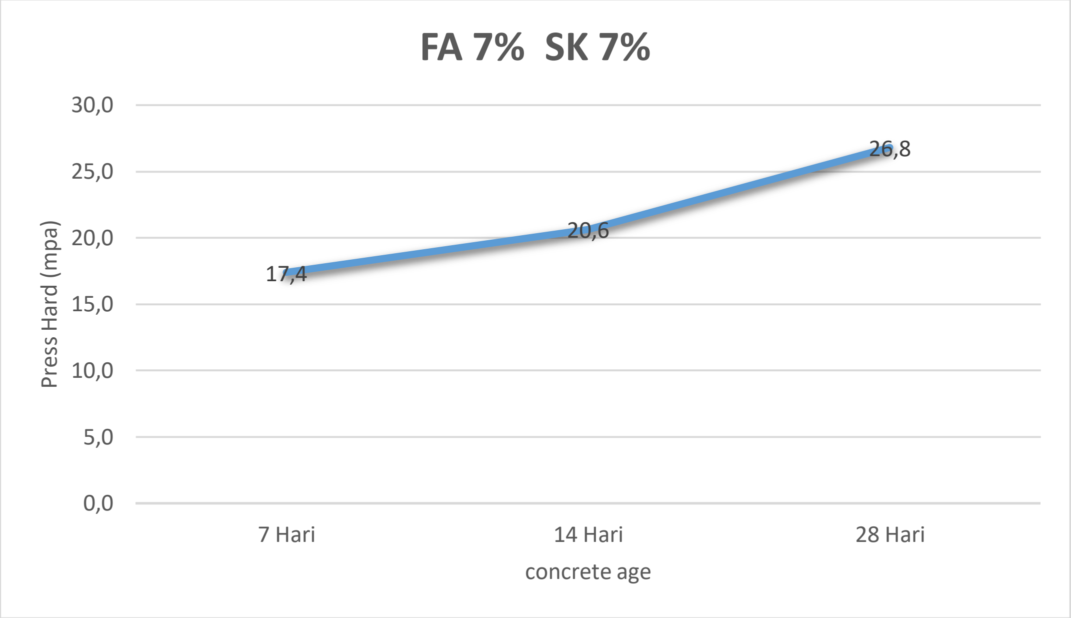
Figure 3. Graph of FA 7% Concrete Test Results and 5% SK



Source: Research Data

Figure 4. Graph of FA 9% and SK 7% Concrete Test Results

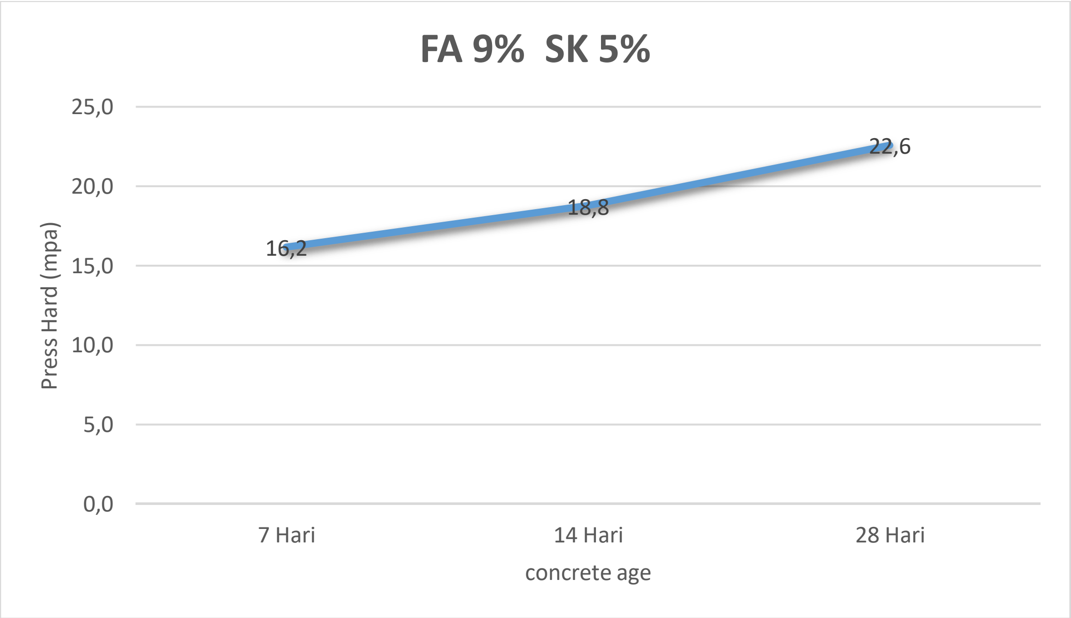
From the table above, it is known that the compressive strength of concrete with the addition of 9% fly ash and 7% glass powder on the 7th day is 14.8 MPa, the 14th day is 17.0 MPa and the 28th day is 19.0 MPa. It can also be seen in the graph that the increase continued to occur at the age of 28 days. With the addition of 9% fly ash and 7% glass powder, the concrete at the age of 28 days reached the K-300 quality, where the concrete entered the class II concrete quality, namely the quality of concrete for structural construction.



Source: Research Data

Figure 5. Graph of FA 7% and SK 7% Concrete Test Results

From the table above, it is known that the compressive strength of concrete with the addition of 7% fly ash and 7% glass powder on the 7th day is 17.4 MPa, the 14th day is 20.6 MPa and the 28th day is 26.8 MPa. It can also be seen in the graph that the increase continued to occur at the age of 28 days. With the addition of 7% fly ash and 7% glass powder, the concrete at the age of 28 days reached the K-300 quality, where the concrete entered the class II concrete quality, namely the quality of concrete for structural construction.



Source: Research Data

Figure 6. Graph of FA 9% and SK 5% Concrete Test Results

From the table above, it is known that the compressive strength of concrete with the addition of 9% fly ash and 5% glass powder on the 7th day is 16.2 MPa, the 14th day is 18.8 MPa and the 28th day is 22.6 MPa. It can also be seen in the graph that the increase continued to occur at the age of 28 days. With the addition of 7% fly ash and 5% glass powder, the concrete at the age of 28 days reached the K-300 quality, where the concrete entered the class II concrete quality, namely the quality of concrete for structural construction.

3.3.Comparison of Compressive Strength of 7 Days Aged Composite Concrete

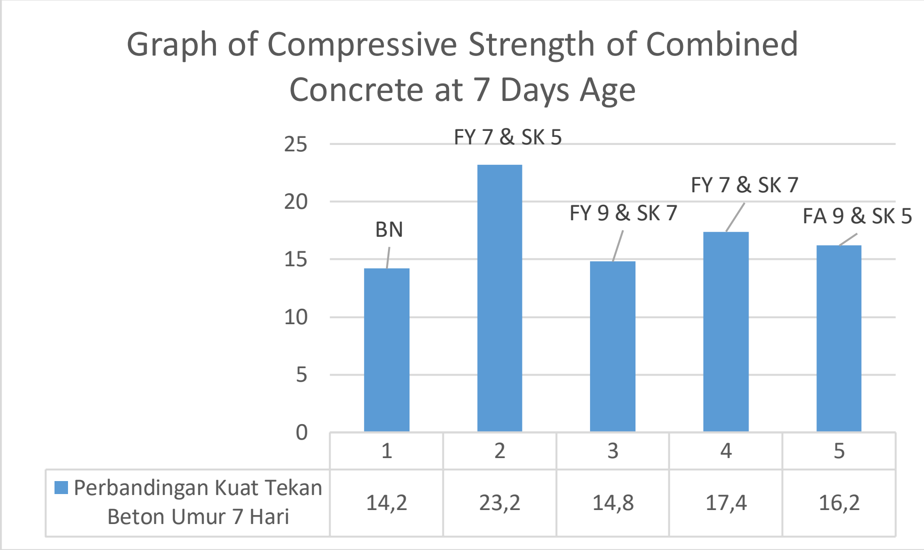


Figure 7. Graph of Compressive Strength of Combined Concrete at 7 Days Age

In the histogram above, it can be seen that the greatest compressive strength of concrete with the addition of fly ash and glass powder at the age of 7 days FA 7% and SK 5% is 23.2 MPa and the smallest concrete compressive strength is FA 9% and 7% which is 14.8 MPa

3.4.Comparison of Compressive Strength of 14 Days Combined Concrete

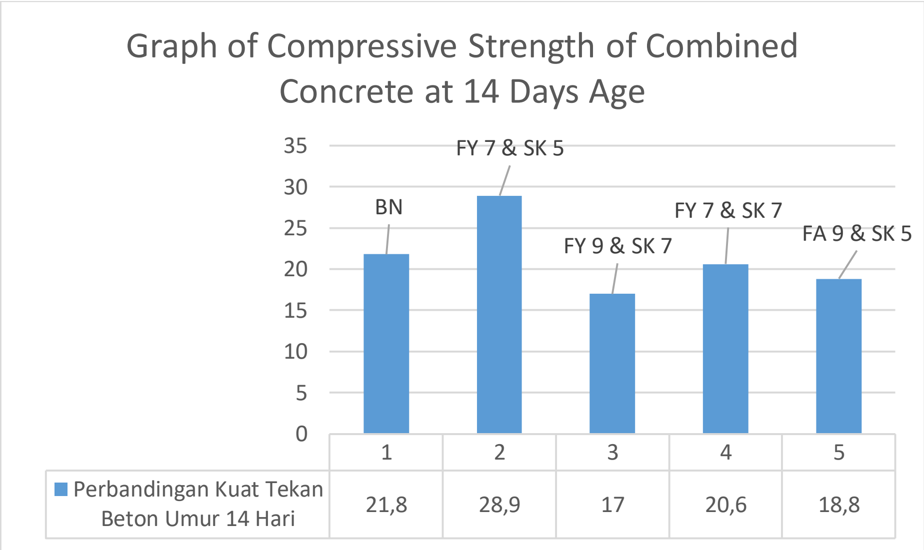


Figure 8. Graph of Compressive Strength of Combined Concrete at 14 Days Age

In the histogram above, it can be seen that the greatest compressive strength of concrete with the addition of fly ash and glass powder at the age of 28 days FA 7% and SK 5% is 28.9 MPa and the smallest concrete compressive strength is FA 9% and 7% which is 17.0 MPa

3.5.Comparison of Compressive Strength of 28 Days Aged Composite Concrete

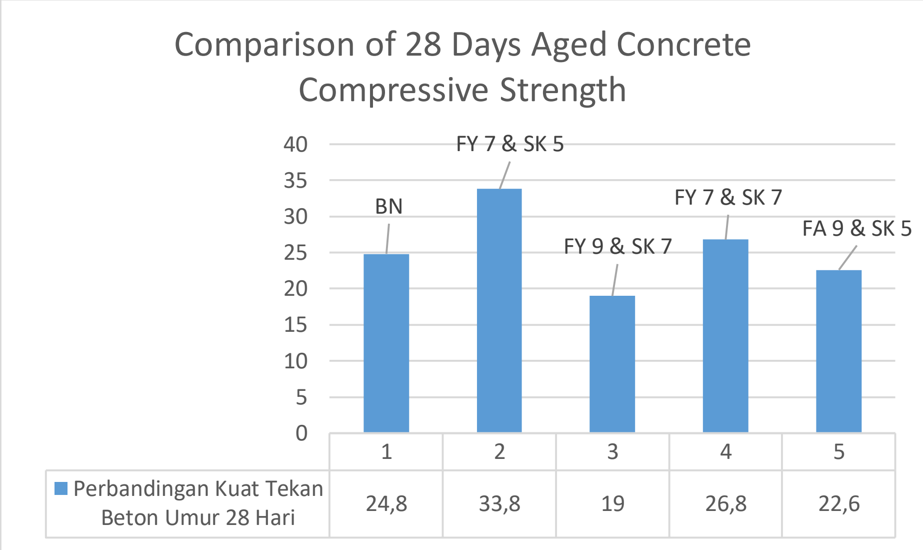


Figure 9. Comparison of 28 Days Aged Concrete Compressive Strength

In the histogram above, it can be seen that the greatest compressive strength of concrete with the addition of fly ash and glass powder at the age of 28 days FA 7% and SK 5% is 33.8 MPa and the smallest concrete compressive strength is FA 9% and 7% which is 19.0 MPa

3.6.Histogram of Compressive Strength of Combined Concrete at Each Test Age

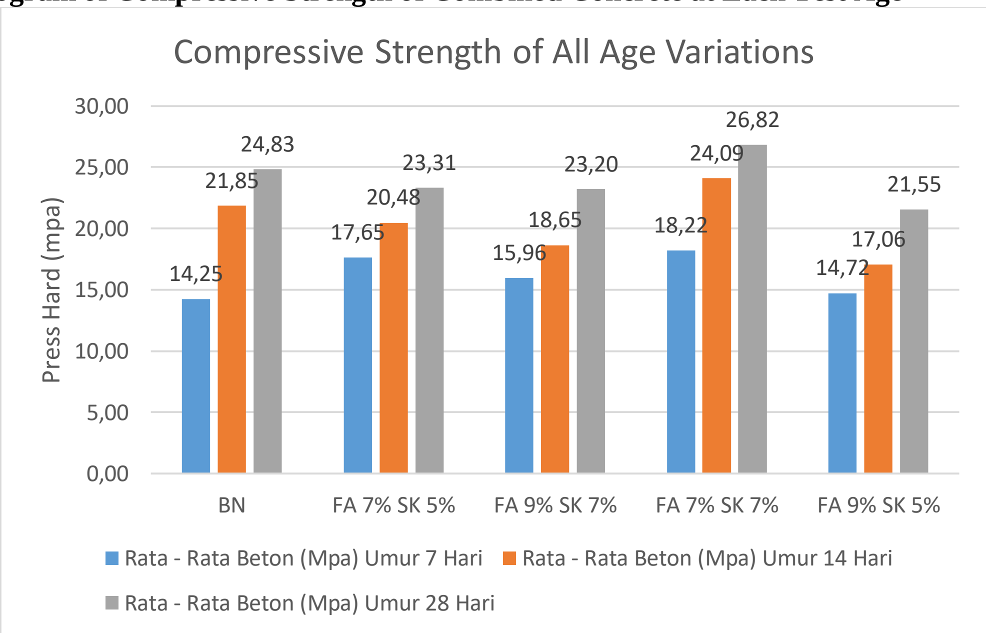


Figure 10. Compressive Strength of All Age Variations

4. Conclusion

1. Based on the Slump value, with BN the slump value is 11 cm, the variation of the mixture of Fly Ash and Glass Powder is in the range of 10 - 14 cm. small so that the level of workability is low.
2. The research proves that Fly Ash and Glass Powder have a great effect on the compressive strength of concrete. Concrete with a mixture of Fly Ash 7% Glass Powder 5% waste obtained a compressive strength at the age of 28 days of 33.82 Mpa. Variation of Normal Concrete sample has a compressive strength of 24.80 MPa at the age of 28 days.

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Cost and Time Performance Analysis on Controlling Contruction Projects with "PERT" Method (Case Study: Airlangga University Parking Building)

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Abstract

In the matter of parking lot problems, there are many ways to solve the problems that always arise from the parking lot with more and more and require a very large land with adequate capacity, therefore one of them is by making a multi-storey parking building. The more development of construction work, the more problems that arise during construction, one of which is project management problems which before the implementation or implementation of development are always planned to determine the schedule, time, and cost needed. In the construction of the Airlangga University multi-storey parking building project, controlling the schedule of the plan should have been carried out in 300 days. With the CPM and PERT methods used, the calculation duration is 340 days with a probability of 99.85% to speed up work and delays. Acceleration is carried out by adding workers so that it will affect costs, so after adding workers with normal costs of Rp. 49.139.000.850,26. and after the addition of manpower to speed up the implementation time with a total cost of Rp 49,227,563,678.83. after accelerating.

Keywords :
Cost, PERT, Time

1. Introduction

A project is an organized effort to achieve important goals, objectives and expectations using the available budget and resources, which are adjusted to a certain time period. One type of project that is often found in our daily lives is the implementation of building construction projects .

In this study, Airlangga University parking building construction project was used as the object of research. Through this research, the researcher wants to know about the time and cost performance in the implementation of the building construction project. The reason the researcher chose the building construction project as the object of research was because of the suspicion that the implementation of the construction project had problems with delays in project completion time so that it also had an impact on project cost overruns.

2. Methodology

Program Evaluation Review Technique (PERT) is a network program that is used to plan and control the duration of a project, the stages of analyzing each activity with three scopes of time, namely optimistic time, pessimistic time, and realistic time to get an estimate of the expected time.

The results of this study become a reference for project implementers regarding building construction project management, which through the results of this study, project implementers can apply the PERT method as a construction project management technique, so as to avoid problems related to construction project time and cost

3. Results and Discussion

3.1. The possibility (probability)

The possibility (probability) of completing a project is an important thing that must be considered in making a work schedule. This is because every project implementation must have constraint factors so that these probabilities can be used to consider the possibility of completing the project faster, slower, or according to a predetermined schedule.

Table 1. The Possibility

NO	Completion Target	Z deviation	Cumulative Normal Distribution	Probability/Probability of the Project to be completed 100%
1	260	-2.9724	0.0015	0.15%
2	271	-2,1550	0.0158	1.58%
3	272	-2,0807	0,0188	1,88%
4	273	-2,0064	0,0228	2,28%
5	274	-1,9321	0,0268	2,68%
6	275	-1,8578	0,0322	3,22%
7	276	-1,7835	0,0375	3,75%
8	277	-1,7091	0,0446	4,46%
9	278	-1,6348	0,0516	5,16%
10	279	-1,5605	0,0594	5,94%
11	280	-1,4862	0,0694	6,94%
12	281	-1,4119	0,0793	7,93%
13	282	-1,3376	0,0918	9,18%
14	283	-1,2633	0,1038	10,38%
15	284	-1,1890	0,119	11,90%
16	285	-1,1147	0,1335	13,35%
17	286	-1,0404	0,1492	14,92%
18	287	-0,9660	0,1685	16,85%
19	288	-0,8917	0,1867	18,67%
20	289	-0,8174	0,209	20,90%
21	290	-0,7431	0,2296	22,96%
22	291	-0,6688	0,2546	25,46%
23	292	-0,5945	0,2776	27,76%
24	293	-0,5202	0,3015	30,15%
25	294	-0,4459	0,33	33,00%
26	295	-0,3716	0,3557	35,57%
27	296	-0,2972	0,3859	38,59%
28	297	-0,2229	0,4129	41,29%
29	298	-0,1486	0,4443	44,43%
30	299	-0,0743	0,4721	47,21%
31	300	0,0000	0,5	50,00%
32	301	0,0743	0,5279	52,79%
33	302	0,1486	0,5557	55,57%
34	303	0,2229	0,5871	58,71%
35	304	0,2972	0,6141	61,41%
36	305	0,3716	0,6443	64,43%
37	306	0,4459	0,67	67,00%
38	307	0,5202	0,6985	69,85%
39	308	0,5945	0,7224	72,24%
40	309	0,6688	0,7454	74,54%
41	310	0,7431	0,7704	77,04%
42	311	0,8174	0,791	79,10%
43	312	0,8917	0,8133	81,33%
44	313	0,9660	0,8315	83,15%
45	314	1,0404	0,8508	85,08%
46	315	1,1147	0,8665	86,65%
47	316	1,1890	0,881	88,10%
48	317	1,2633	0,8962	89,62%
49	318	1,3376	0,9082	90,82%
50	319	1,4119	0,9207	92,07%
51	320	1,4862	0,9306	93,06%
52	321	1,5605	0,9406	94,06%
53	322	1,6348	0,9484	94,84%
54	323	1,7091	0,9554	95,54%
55	324	1,7835	0,9625	96,25%
56	325	1,8578	0,9678	96,78%
57	326	1,9321	0,9732	97,32%
58	327	2,0064	0,9772	97,72%
59	328	2,0807	0,9812	98,12%

60	329	2,1550	0,9842	98,42%
61	330	2,2293	0,9864	98,64%
62	331	2,3036	0,9893	98,93%
63	332	2,3779	0,9911	99,11%
64	333	2,4523	0,9929	99,29%
65	334	2,5266	0,9941	99,41%
66	335	2,6009	0,9953	99,53%
67	336	2.6752	0.9962	99.62%
68	337	2.7495	0.9969	99.69%
69	338	2.8238	0.9976	99.76%
70	339	2.8981	0.9981	99.81%
71	340	2.9724	0.9985	99.85%

From the results of the analysis above, it can be seen that:

1. The probability that the project can be completed in 260 days is 0.15%.
2. The probability that the project can be completed in 300 days is 50%.
3. The probability that the project can be completed in 340 days is 99.85%.

This parking building construction project has activities that must be carried out systematically. After processing the data, the data obtained that the probability of project completion is 340 days with a probability of 99.85% possibility, therefore a method of accelerating the addition of workers is needed because the project stated in the contract document for the completion of the project is 300 days, by using the alternative of adding workers to the track activities. critical will make activities more quickly completed. In this case, the additional manpower is carried out by the project implementer according to the needs in the field. from the addition of labor there is an increase in the cost of the project.

3.2. Calculation Crash Duration and Crash Cost

Calculation Crash Duration and Crash Cost then carried out for the activities of all project activities, so that if recapitulation will get the value as following:

Tabel 2. Calculation Crash Duration and Crash Cost

No	Activity	Normal Duration (Day)	Normal Cost (Rp)		Crash Duration (day)	Crash Cost (Rp)	
1	a.1	30.7	Rp	44,359,786.46	25.6	Rp	48,097,715.04
2	a.2	16.3	Rp	21,949,843.20	13.6	Rp	23,931,814.63
3	a.3	13.3	Rp	24,369,492.40	11.1	Rp	25,986,363.82
4	a.4	17.9	Rp	40,747,864.56	17.9	Rp	40,747,864.56
5	a.5	3.6	Rp	3,129,374,80	3.6	Rp	3 ,129,374, 80
6	a.6	14.0	Rp	44,752,500.00	14.0	Rp	44,752,500.00
7	b.1	45.7	Rp	127,615,924.12	45.7	Rp	127,615,924.12
8	b.2	31.4	Rp	30,085,949.32	26.2	Rp	33,910,806.46
9	b.3	30.7	Rp	387,198,576.93	25.6	Rp	390.936.505.50
10	b.4	21.3	Rp	27,418,127.04	21.3	Rp	27,418,127.04
11	b.5	13.0	Rp	17,773,587.05	10.8	Rp	19,355,687.05
12	c.1	110.9	Rp	11,270,434,768,64	110.9	Rp	11 ,270,434,768, 64
13	c.2	96.1	Rp	2,213,864,225,70	96.1	Rp	2 ,213,864,225, 70
14	c.3	90.0	Rp	67,263,489.25	90.0	Rp	67,263,489.25
15	c.4	84.3	Rp	2,126,538,256.58	84.3	Rp	2,126,538,256.58
16	c.5	95.9	Rp	760,187,704.53	95.9	Rp	760,187,704.53
17	c.6	88.3	Rp	1,161,381,099.39	88.3	Rp	1,161,381,099.39
18	c.7	76.0	Rp	1.042.015.286.16	76.0	Rp	1.042.015.286.16
19	c.8	80.3	Rp	3,667,379,488.01	80.3	Rp	3,667,379,488.01
20	c.9	46.6	Rp	108,621,701.63	38.8	Rp	114,289,444.49
21	c.10	76.7	Rp	554,786,237.07	76.7	Rp	554,786,237.07
22	c.11	36.0	Rp	64,212,328.93	30.0	Rp	68,593,528.93
23	c.12	36.0	Rp	128,561,346.56	30.0	Rp	132,942,546.56
24	d.1	45.4	Rp	1,161,381,099.39	45.4	Rp	1,161,381,099.39
25	d.2	45.4	Rp	1.042.015.286.16	45.4	Rp	1.042.015.286.16
26	d.3	75.4	Rp	1,991,941,513.50	75.4	Rp	1,991,941,513.50
27	d.4	34.1	Rp	108,621,701.63	28.5	Rp	112,776,887.35
28	d.5	52.1	Rp	406.036.297.04	52.1	Rp	406.036.297.04
29	d.6	52.1	Rp	1,101,745,754.39	52.1	Rp	1,101,745,754.39

30	d.7	37.3	Rp	128,561,346.56	31.1	Rp	133.099.017.99
31	e.1	45.4	Rp	1,161,381,099.39	45.4	Rp	1,161,381,099.39
32	e.2	45.4	Rp	1.042.015.286.16	45.4	Rp	1.042.015.286.16
33	e.3	75.4	Rp	1,991,941,513.50	75.4	Rp	1,991,941,513.50
34	e.4	34.1	Rp	108,621,701.63	28.5	Rp	112,776,887.35
35	e.5	52.1	Rp	405,951,248.23	52.1	Rp	405,951,248.23
36	e.6	52.1	Rp	1,101,745,754.39	52.1	Rp	1,101,745,754.39
37	e.7	37.3	Rp	128,561,346.56	31.1	Rp	133.099.017.99
38	f.1	45.4	Rp	1,161,381,099.39	45.4	Rp	1,161,381,099.39
39	f.2	45.4	Rp	1.042.015.286.16	45.4	Rp	1.042.015.286.16
40	f.3	52.1	Rp	1,991,941,513.50	52.1	Rp	1,991,941,513.50
41	f.4	37.3	Rp	108,621,701.63	31.1	Rp	113.159.373.06
42	g.1	26.7	Rp	6,586,790.66	26.7	Rp	6,586,790.66
43	g.2	59.6	Rp	168,044,021.53	49.6	Rp	175,293,864.39
44	h.1	14.7	Rp	5,124,536.20	12.3	Rp	6,915,264.77
45	h.2	40.1	Rp	15,500,149.62	33.5	Rp	20,385,535.34
46	h.3	41.6	Rp	416,139,045.58	34.6	Rp	421,198,288,44
47	i.1	53.6	Rp	1,275,360,201.31	53.6	Rp	1,275,360,201.31
48	i.2	51.3	Rp	845,961,580,36	51.3	Rp	845,961,580,36
49	i.3	51.3	Rp	851,418,162.76	51.3	Rp	851,418,162.76
50	i.4	47.7	Rp	221.3323.620.12	39.8	Rp	227,130,448,69
51	j.1	98.6	Rp	272,611,940.00	98.6	Rp	272,611,940.00
52	j.2	76.0	Rp	19,310,955,80	76.0	Rp	19 ,310,955, 80
53	j.3	76.7	Rp	485,507,688,70	76.7	Rp	485,507,688,70
54	j.4	97.3	Rp	1,225,927,068.10	97.3	Rp	1,225,927,068.10
55	j.5	107.7	Rp	131.564.716.00	107.7	Rp	131.564.716.00
56	j.6	97.7	Rp	4,907,800,000	97.7	Rp	4,907,800,000
57	j.7	100.4	Rp	113,304,160.00	100.4	Rp	113,304,160.00
58	j.8	102.6	Rp	375,963,386.00	102.6	Rp	375,963,386.00
59	j.9	85.0	Rp	224,736,600,000	85.0	Rp	224,736,600,000
60	j.10	76.0	Rp	26,847,000.00	76.0	Rp	26,847,000.00
61	j.11	73.1	Rp	73.803.840.00	73.1	Rp	73.803.840.00
62	j.12	76.1	Rp	62,450.000,00	76.1	Rp	62 ,450.000, 00
63	j.13	47.6	Rp	158,553,100,00	47.6	Rp	158,553,100,00
64	j.14	47.6	Rp	434,000,000.00	39.6	Rp	439,789,442.86
65	j.15	49.3	Rp	144,955,000.00	49.3	Rp	144,955,000.00
66	j.16	47.7	Rp	183.550.000.000,00	47.7	Rp	183.550.000.000,00
67	j.17	47.0	Rp	45,342,500.00	47.0	Rp	45,342,500.00
68	j.18	47.7	Rp	217,846.590.00	47.7	Rp	217,846.590.00
69	j.19	40.1	Rp	106,644,290.00	40.1	Rp	106,644,290.00
70	j.20	39.7	Rp	89,323,600,000	39.7	Rp	89,323,600,000
71	j.21	42.3	Rp	817,265,000.00	35.2	Rp	822,411,171.43
TOTAL	Rp	49,139,000,850,26	TOTAL	Rp	49,227,563,678.83		

Thus, the addition of workers in the construction of the Airlangga University Surabaya parking building project can speed up work, especially in the critical path which has no time to delay in its accelerated work. The following is the total cost of all costs that must be incurred by the project, namely from the normal value of Rp. 49,139,000,850.26 to Rp. 49,227,563,678.83. From the duration of the project calculation, the possible length of work is 340 days with a probability of 99.85% , the rest is 40 days later than the 300 day work duration contract. can be accelerated with the addition of handyman labor as calculated above.

4. Conclusions and recommendations

4.1. Conclusion

Based on the writing that has been done in the construction project of the Parakeet Building, Universitas Airlangga Surabaya, it is concluded as follows:

1. By using the PERT method, the Airlangga University Surabaya Parking Building construction project can be completed the fastest in 260 days with a probability of 0.15%, at the latest it can be completed in 340 days with a probability of 99.85%, and the most likely to be completed in 300 days with probability probability 50%

2. By ensuring that the acceleration is carried out with a time duration of 340 days with a probability of 99.85%, so that activities on the critical path can be carried out by adding labor which is obtained at a normal cost of Rp. 49,139,000,850,26 while the costs incurred by the project after the addition of a handyman or time acceleration with a total cost of Rp 49,227,563,678.83 after being accelerated.

4.2. Suggestion

For further research, it is necessary to calculate the inhibiting factors and other accelerations to get a more optimal time in it in order to reduce the inhibiting factors in its implementation.

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Soil Stabilization (Subgrade) for Ainaro Lot 2 Road Paper, Timor – Leste

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Abstract

Soil conditions in Timor Leste vary greatly in terms of grain and bearing capacity. The soil found at the laulara solerema location is clay because it can be seen from various types of samples and the results of analysis in the laboratory can determine the quality of the soil and the type of soil. This study aims to determine the characteristics of clay soil in Laura – Solerema as a road pavement material, especially the soil foundation layer.
The method used in determining the mixture through several tests, among others, sieve analysis test, compaction test (standard proctor) CBR (California Bearing Ratio) test, Swelling, Atterberg limit test and compaction test. The results of testing the clay soil in Laura – Solerema partially have values that do not meet the requirements as road pavement materials, especially subgrade layers. The stabilization study aims to study and determine the results of soil test data, to be able to determine the optimum moisture content (OMC) and dry volume (MDD), and the Atterberg limit with CBR values of 95% and 100%.

Keywords :
Clay, Sand and Cbr

1. Introduction

Subgrade is the most important part in road construction, and the problem that occurs in road construction projects is the reduction of road embankment soil, causing damage to asphalt and subsidence of embankment caused by inadequate soil carrying capacity, and excessive soil water content. A common problem that is often experienced in the implementation of road construction is that subgrade soil is not always found that has sufficient capacity to withstand the traffic load above. Roads in laulara – solerema are in some locations influenced by the nature of the soil layer that composes them, including the subgrade of the road made of earth. Soil as the subgrade layer used can be in the form of accumulated soil or unique soil. The nature of the road is strongly influenced by the asphalt layer that forms the road construction. From these problems, what can overcome these problems is soil improvement (stabilization).

Subgrade stabilization is an efficient decision in recognizing good road quality and reducing construction work costs. Clay is a silicate striped mineral molecule that measures under 4 micrometers. Clay contains fine silica or aluminum, these components, silicon, oxygen, and aluminum being the most abundant components. In the development of asphalt roads, the qualityof the soil that is not empowered must be improved (improved) so that it can work at the carrying capacity limit (subgrade) to improve the situation.

Soil stabilization in Sta. 14+070-15+370 B/S from the results of the study identified that it must replace or improve the base soil (subgrade) with accurate material to withstand the load to be built on the road, especially on highways. Research conducted at Sta. 14+070-15+370 B/S on subgrade soils, using the AASTHO T199-01/180-01 method for sieve, Liquid Limit(LL) and Plastic Limit(PL) analysis testing. CBR and SWELLING data testing uses the AASHTO T 193-99 method.

According to Lestari (2013) the asphalt layer generally consists of a layer of asphalt that is arranged from the bottom up, especially the subgrade layer, sub-base layer, base layer and surface layer. course). Subgrade is the first layer of soil in the road structure area. The soil must have adequate bearing capacity to withstand the overburden.

1.2. Formulation of the problem

Based on the above basis, which will be discussed in this study are:

1. How is the land classification in Ainaro Lot 2, Timor - Leste?
2. What is the CBR value of the original and stabilized soil with added materials?

1.3. Scope of problem

The problem limitations of this research are:

1. This research must be carried out in a laboratory
2. Improvement of soft soil in an effort to use it for road material.

3. Lowering the plastic index value of the soil and increasing the CBR . value

1.4. Research purposes

1. To determine the effect of the added additives on the physical properties of the soil.
2. Comparing the CBR value of unstabilized clay with stabilized soil.
3. Knowing the percentage of the optimum water content of the additive that has been increased with additional ingredients.

1.5. Benefits of research

The benefits of this research are:

1. As a science of soil stabilization for the improvement of the subgrade of a road construction, especially some flexible pavements.
2. From the results of the research and calculations carried out, it is expected to be able to provide an overview of the increase in the carrying capacity of the original soil with a mixture of mechanical materials.

2. Basic Theory Used

2.1. Soil Stabilization

Soil improvement is an approach to improve or change the description of a less positive subgrade condition as far as the bearing capacity of the soil is used as the basis for its development, it is believed that the subgrade properties can be better. There are several soil adjustment strategies that are generally used with the ultimate goal of working on subgrade properties of poor quality. These techniques combine mechanical adjustment and substance adjustment. This mechanical adjustment is proposed to get the entire soil evaluated so that the subgrade can meet the predetermined details. Adjustment by mechanical means is generally carried out by mixing different soil types,

2.2. Soil classification according to AASHTO.

Subgrade CBR and DCP Test Results The purpose of the laboratory CBR test and in-situ DCP verification is to provide information about the mechanical properties for the upper from the subsurface. To convert the results of DCP to CBR, the following equation developed by Kleyn & Van Heerden (1983) was used:

1. CBR logs = 2.628 -1.273 log

CPIWhere:

2. CPI or cone penetration index is the penetration measured in terms of mm / stroke.

The equation is that if the penetration per stroke becomes very low (less than 3 mm / stroke) the CBR exceeds 100 and becomes negligible so these values are recorded as CBR 100. If the DCP results generally support field density and CBR, there is less emphasis on DCP results. when parts of the site are not suitable for DCP testing, with a number of tests carried out on large gravel sized materials.

Isolated values (CBR ~ 100) were excluded from data analysis.

3. Research of Soil Mechanical Properties

3.1. Soil Compaction Test (Standard Proctor)

The research material used was a soil sample from the laulara – solerema area of Timor Leste. The soil samples were brought to perform several tests to obtain soil properties, soil type, water content, and others. The standard compactor press is used to compact the soil. Proctor there is a relationship between water content and dry volume weight of the soil. There are various types of soil that have an optimum moisture content value to achieve maximum dry volume weight.

According to Hardiyatmo (2010), the motivation behind compaction is as follows:

1. Increase the shear strength of the soil,
2. Reduce compressibility,
3. Lowering the penetrating power, and
4. Reduces volume changes due to changes in water content.

3.2. Atterberg Test

Atterberg is a test method to determine the nature of fine-grained soil (clay or silt) by providing different water content for each sample to be tested.

1. Place a portion of the prepared sample into the liquid limit device cup at the point where the cup rests on the base and spread it so that it is 10mm deep at its deepest point. Form a horizontal surface above the ground. Take care to remove any air bubbles from the soil specimen. Store unused specimens in storage containers.

2. Make a groove in the soil by drawing the groove making tool, tilting the edge forward, through the soil from the top of the cup to the bottom of the cup. When forming the groove, hold the tip of the grooving tool against the cup surface and keep the tool perpendicular to the cup surface.
3. Lift and drop the cup at a rate of 2 drops per second. Continue cranking until the two halves of the soil specimen meet each other at the bottom of the groove. The two halves should meet a distance of 13 mm (1/2 inch).
4. Record the number of drops needed to close the groove.
5. Remove a piece of soil and determine its water content, w.
6. Repeat steps 1 through 5 with soil samples with slightly higher or lower moisture content. Whether water should be added or removed depends on the number of strokes required to cover the forest in the previous sample.

3.3. Swelling

Swell test is the measurement of samples immersed in water to be carried out after reading and before reading to be able to know the water that enters the sample or can know the thickness of the soaked soil.

3.4. Test Cbr (California Bearing Ratio)

According to Sukirman (1995) the California Bearing Ratio (CBR) test determines the strength of the soil or a combination of compacted soil at a certain moisture content. The CBR is the ratio of the embankment required for 0.1"/0.2" soil sample infiltration to the embankment holding standard crushed rock at the 0.1"/0.2" entrance. The CBR value is the proportion (in percent) between the pressing factor required to infiltrate the ground with a 3-inch circular cylinder at a speed of 0.05 inches/min to the pressing factor required to insert a certain standard material and the intentional load required to enter.

1. This test is isolated into two, in particular;
   a. CBR testing without watering (Unsoaked) with restore for 1 day, 3 days and also 7 days.
   b. CBR drenching test (Soak) which is first restored for 3 days and then submerged in water for 4 days to determine the benefits of expanding. After watering, a soil test can be attempted for CBR. CBR testing is carried out on the first soil, soil + sand

4. Research Methods

4.1. Research Flowchart

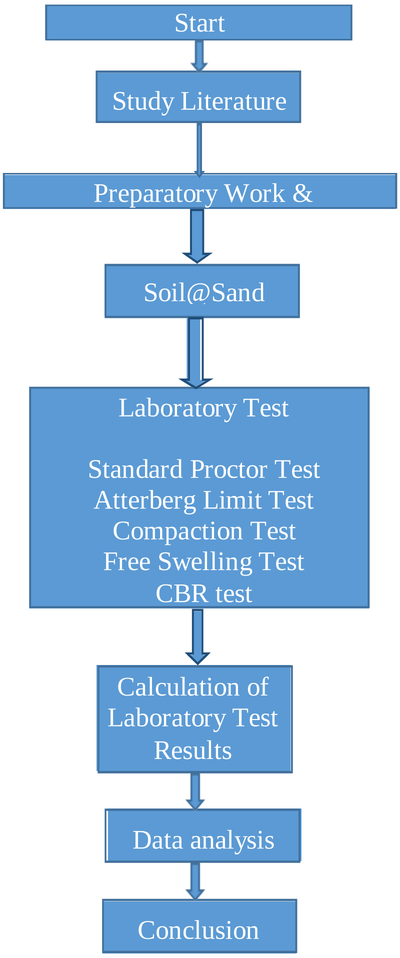

Figure 1. Research Flowchart

4.2. Analysis and Discussion

Before planning the structure of a highway building, the first step is to inspect the soil condition. Soil inspection is carried out to determine whether the soil is in accordance with the predetermined classification. In the research in the laboratory in Cotolau, the test results used only sieve analysis and Atterberg limits. From these results, there is a Liquid Limit value of 38.2%, Plastic Limit 28.6% and Plastic Index 9.6% and the results obtained that the most dominant type of material is clay or group A-6 classification. Can be seen in table 1.

Table 1. Soil classification for road subgrade

General classification	Silt – Clay (More than 35% of all soil samples passed the No 200 sieve)			
Group classification	A-4	A-5	A-6	A-7 A-7-5 A-7-6
Sieve analysis (% pass) No. 10 No. 40 No. 200	Min 36	Min 36	Min 36	Min 36
The nature of the fraction that passed the No. sieve. 40 Liquid limit (LL) Plasticity Index (PI)	Min 40 Max 10	Max 41 Max 10	Max 40 Min 11	Min 41 Min 11
The most dominant type of material	silty land		loamy soil	
Appraisal as subgrade material	Ordinary to ugly			

4.2.1. Testing the Mechanical Properties of Clay Soil

Based on research that has been carried out in the Cotolau laboratory, Dili district, the results are in the form of mechanical properties that describe the strength and stabilization of the soil as listed in table 2.

Table 2. the results of testing the mechanical properties of clay

Soil Mechanical Properties	Score
Maximum volume weight (MDD)	1,874
Optimum moisture content (OMC)	15,000
Plastic Index	11,1
CBR	5.8
Development potential	1.99

Based on table 2 the results obtained and the Proctor compaction test (standard) include the optimum moisture content (OMC) for organic clay of 15,000 and the maximum dry volume weight (MDD) value of 1,874 g/cm2. The value of the optimum water content for the manufacture of CBR specimens. The method used for CBR testing is soaked CBR with an immersion period of 4 days. This method is used to simulate the worst conditions in the field when inundation occurs for a long period of time. The CBR value for organic clay is 5.8 and according to the planning carried out in Laulara – Solerema, it is explained that the CBR value for the subgrade is 8%. Therefore, based on the research results, this organic clay does not meet the requirements as a road subgrade.

5. Conclusions and Recommendations

5.1. Conclusion

From the results of research and analysis that has been done, it can be concluded as follows:

1. Based on the physical properties of the soil from the results of soil investigations from Cotolau Laboratory, it has a dry volume weight (MDD) of 1.874 g/cc, optimum water content (OMC) 15,000 % Liquid Limit 38.2% , Plastic Limit 27.6%, so that the plasticity value index, PI, obtained 11.1% with soil classification A-6 and clayey materials.
2. The 95% CBR value for native land is 5.8% and for 100% with a value of 6.8%, the CBR value for native land does not meet the standards because the standard CBR value for native land is 8.0%. When stabilizing the soil with sand, the CBR value meets the standard, with a mixture of sand. The 95% CBR value for sand is 20% and for 100% it is 35.2%.

5.2. Development Suggestions

In light of the tests and conversations above, some ideas for additional checks can be taken as follows:

1. Additional checks are needed in dissecting the bearing capacity of the soil using different techniques with different loads to determine soil settlement.

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