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Contents

- Page 194-209
  Analysis of Streamflow Response to Changing Climate Conditions Using SWAT Model
  Han Thi Oo, Win Win Zin, Cho Cho Thin Kyi

- Page 210-224
  Environmental and Economic Analysis of Selected Pavement Preservation Treatments
  Kelvin Zulu, Rajendra P. Singh, Farai Ada Shaba

- Page 225-237
  Behaviour of Soft Clayey Soil Improved by Fly Ash and Geogrid under Cyclic Loading
  Hussein H. Karim, Zeena W. Samueel, Adel H. Jassem

- Page 238-257
  Effective Utilization of Municipal Solid Waste as Substitute for Natural Resources in Cement Industry
  Abdur Rehman, Kashif Ali Khan, Tayyaba Hamid, Hassan Nasir, Izhar Ahmad, Muhammad Alam

- Page 258-264
  Laboratory Investigation on Interaction of the Pile Foundation Strengthening System with the Rebuilt Solid Pile-Slab Foundation
  Y. A. Pronozin, M. A. Stepanov, D. V. Rachkov, D. N. Davlatov, V. M. Chikishev

- Page 265-275
  Compressive Strength and Elastic Modulus of Slurry Infiltrated Fiber Concrete (SIFCON) at High Temperature
  Ali Mudhafar Hashim, Mohammed Mansour Kadhum

- Page 276-284
  Weather Impact on Passenger Flow of Rail Transit Lines
  Yongqing Guo, Xiaoyuan Wang, Qing Xu, Shanliang Liu, Shijie Liu, Junyan Han

- Page 285-303
  Assessment of SMC Frames under Different Column Removal Scenarios
  Mariam Mohammed Ehab, Mina Mokhtar Maxi

- Page 304-317
  Assessment of Moisture Susceptibility for Asphalt Mixtures Modified by Carbon Fibers
  Huda Qasim Mawat, Mohammed Qadir Ismael

- Page 318-325
  Hydrochemical Characterisation of Groundwater Quality: Merdja Plain (Tebessa Town, Algeria)
  Baazi Houria, Kalla Mahdi, Tebbi Fatima Zohra
Contents

- Page 326-343
  A Comprehensive Numerical Study on Building-Excavation Interaction
  Arman Maddah, Abbas Soroush

- Page 344-362
  Research on Application of Buckling Restrained Braces in Strengthening of Concrete Frame Structures
  Niyonyungu Ferdinand, Zhao Jianchang, Yang Qingqiang, Guobing Wang, Xu Junjie

- Page 363-374
  The Fire Exposure Effect on Hybrid Reinforced Reactive Powder Concrete Columns
  Hasanain A. Shubbar, Nameer A. Alwash

- Page 375-383
  Fatigue Resistance Models of Structural for Risk Based Inspection
  Sergei Belodedenko, V. Hanush, A. Baglay, O. Hrechanyi

- Page 384-401
  Particle Swarm Optimization Based Approach for Estimation of Costs and Duration of Construction Projects
  Tarq Zaed Khalaf, Hakan Çağlar, Arzu Çağlar, Ammar Nasiri Hanoon

- Page 402-417
  Cementitious, Pozzolanic And Filler Materials For DSM Binders
  John Kok Hee Wong, Sien Ti Kok, Soon Yee Wong
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Analysis of Streamflow Response to Changing Climate Conditions Using SWAT Model

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Abstract

The understanding of climate change is curial for the security of hydrologic conditions of river basins and it is very important to study the climate change impacts on streamflow by analyzing the different climate scenarios with the help of the hydrological models. The main purpose of this study is to project the future climate impact on streamflow by using the SWAT model. The multi-model projections indicated that Upper Ayeyarwady River Basin is likely to become hotter in dry season under low rainfall intensity with increasing temperature and likely to become wetter but warmer in both rainy and winter season because of high rainfall intensity with increased temperature in future. The impact of climate change scenarios is predicted to decrease the annual streamflow by about 0.30 to 1.92% under RCP2.6, 5.59 to 7.29% under RCP4.5 and 10.43 to 11.92% under RCP8.5. Based on the change in high and low flow percentage with respect to the baseline period, the difference between high and low flow variation range will increase year by year based on future scenarios. Therefore, it can be concluded that it may occur more low flow in the dry season which leads to increase in water scarcity and drought and more high flow in the wet season which can cause flooding, water insecurity, stress, and other water-related disasters.

Keywords: Climate Change; SWAT Model; Streamflow; High and Low Flow.

1. Introduction

The assessment of climate change impact on streamflow is one of the most interesting issues in hydrological research [1]. Changes in air temperature and precipitation cause a major impact on the hydrologic cycle directly and indirectly and moreover, the water resources [2]. Climate change altering the amount, intensity, form, and timing of precipitation as well as the rate of evapotranspiration also affects hydrological regimes by affecting the volume, peak rate, and timing of river flow [3]. For studying the impact on the regional water resource availability, the estimation of changes in river flow is the most common and is considered for decision-making processes in water-resource management [4].

Myanmar is situated in the tropical climate region with three dominant seasons: the hot season (16 February to May), the wet season (June to September), and the cold season (October to 15 February) [5] and a region that is highly vulnerable to impacts from climate change. There are about 60 rivers in Myanmar [6], the country’s largest main river is Ayeyarwaddy and it is an important commercial waterway used for trade and transport. The Ayeyarwaddy River is divided into the upper and lower parts with the river confluence with the Chindwin River [7]. Upper Ayeyarwaddy river basin is one of the major river basins in Myanmar and consists of Central Dry Zone and the Northern Hilly Region. The central dry zone area is known as the “oil pot” of the country and the economic growth of the country through agricultural development is essential in prenatal economic life. However, current climate change effects such as high temperature,
scarce rainfall, etc. are now threatening agricultural crops and farmers’ livelihood. So, climatic condition in dry zone region is the key factor for the development of the agriculture and farmers’ livelihood [8]. In the northern hilly region, it is already experiencing the problems of flood and heavy rains [9] and this effect may be more severe in the future due to climate change [10]. Therefore, information about the future streamflow related to climate change for this river plays as a fundamental role. It is essential to quantify and understand climate changes in the Upper Ayeyarwady river basin and those likely to occur over the coming century. This is also a starting point that Myanmar’s stakeholders can use to plan for more summer monsoon rainfall in agriculture, hydropower, conservation areas, dams, flood management and so on [11].

It is important to understand Information derived from Global Climate Models (GCMs) and general characteristics of GCMs for assessing both past and future likely changes in climate scenarios and for assessing the climate change impact on hydrological analysis [12]. Future climate scenarios were produced based on a statistical relationship between climate variables at one or more GCM grid points with the variable of interest at a particular station [13]. Scenarios are images of how the world is likely to evolve in the future in terms of greenhouse gas [14]. In this study, climate information obtained from Representative Concentration Pathways (RCPs) is used to forecast future hydrological changes. There are so many hydrological models for understanding the impact of climate change on the nature of hydrological flow and for calculation of water discharge more accurately, easily and quickly than the traditional measurement method. Soil and Water Assessment Tool (SWAT) is one of the most popular modelling software for assessing hydrologic impacts. The main objective of this study is to forecast the impact of future climate projections on streamflow of the future period by using the projections of precipitation and temperature based on outputs of selected suitable climate models from downloaded 10 GCMs under RCP Scenarios across the Upper Ayeyarwaddy River Basin. The structure of this article is organized as follows: study area descriptions are presented in Section 1. Methodological framework is described in Section 2 and materials and methods of this study are described in Section 3. Section 4 presents the results and discussions of this study. Finally, conclusion is described in Section 5.

1.1. Study Area

In this study, climate change impacts on the water sector highlight the Upper Ayeyarwaddy river basin which is covering about 60% of the total area of Myanmar and originates at the confluence of the N’Mai Hka and Mali Kha rivers. The Upper Ayeyarwaddy is situated at 20˚22’ - 28˚50’ north latitude and 94˚56’ - 98˚42’ east longitude [15] and covered by Kachin State, Mandalay Division, the western part of Shan state and Southeastern part of Sagaing Division as shown in Figure 1. The outlet of the whole basin was selected at Sagaing and the watershed area for this Upper Ayeyarwaddy River is 152,264 km².

![Figure 1. Location of the study area](image)
2. Methodological Framework

The objective of this study has combined two aspects: firstly, assessment of climate change impacts on the climate variables (rainfall and temperature) and secondly, assessment of the response on the river’s hydrologic system of climate variables. The Methodological Framework of this study is shown in Figure 2 and involves: (I) spatial and climate data preparation into SWAT format, (ii) model setup, including watershed delineation and Hydrologic Response Units (HRUs), (iii) model calibration and validation, and (iv) assessment of future climate change impacts on streamflow. SWAT model which is ArcGIS extension, ArcSWAT 2012 version was downloaded from the United States Department of Agriculture (USDA) website. SWAT is a partially distributed model and required digital elevation model (DEM), land use and soil map which are basic modeling requirements and daily weather data. Calibration and Validation were performed using the SWAT CUP program. At this stage, several hydrological model parameters were adjusted for achieving the best fit between the simulated and measured flow at the monitoring station. Finally, climate change impact on future streamflow is projected by using the SWAT model based on meteorological changes under climate change projection.

Figure 2. Methodology framework for the assessment of climate change impacts on future flows of the Upper Ayeyarwady River Basin
3. Materials and Method

3.1. Global Climate Models (GCMs)

GCMs are the primary tools that provide reasonably accurate global, hemispheric, and continental-scale climate information and are used to understand present and future climate scenarios under increased greenhouse gas concentrations [13]. The CMIP5 is a newly developed data archive and contains a great number of model output enhance the understanding of climate processes and their effects. These data will provide a basis of the Intergovernmental Panel on Climate Change (IPCC) Fifth Assessment Report (AR5). GCMs are models to generate a description of the state of the atmosphere and produce most of the meteorological variables, such as wind speed, relative humidity, rainfall, surface air temperature and solar radiation [16].

3.2. Representative Concentration Pathways

RCP based projections were used in the most recent IPCC Fifth Assessment Report (AR5). According to the change in radiative forcing by 2100, there are four RCPs. RCP 2.6 is representative of scenarios that lead to very low greenhouse gas concentration levels [17]. RCP 4.5 and RCP 6 are stabilization scenarios that lead to intermediate greenhouse gas concentration levels. RCP 8.5 is representative of scenarios that lead to high greenhouse gas concentration levels [18]. The future projections are based on the future radiative forcing of the atmosphere. Three of the RCPs: the low emission scenario (RCP2.6), the mitigation scenario (RCP4.5) and the high emission scenario (RCP8.5) which is characterized by increasing greenhouse gas emissions are used in this study.

3.3. Meteorological and Hydrological Data Collection

Weather data such as daily precipitation and temperature data of all fourteen stations within Upper Ayeyarwaddy River Basin are used in this study form 1981 to 2015 and these are acquired from the Department of Meteorology and Hydrology (DMH). Other data such as wind speed, solar radiation and relative humidity from the period 1981 to 2013 are collected from Global Weather Data for SWAT Website. The data availability period of the Sagaing hydrological station which is the outlet station of the Upper Ayeyarwaddy River Basin is 1991 to 2015.

3.4. Soil and Water Assessment Tool (SWAT) Model

Hydrological models are becoming more and more widespread, mainly due to their capacity to simulate the impact of environmental changes on water resources [19]. The Soil and Water Assessment Tool abbreviated as SWAT is a basin-scale model that was developed by the United States Department of Agriculture (USDA) – Agriculture Research Service (ARS) [20]. The SWAT model simulates hydrology as a two-component system, composed of land and channel hydrology. The land portion of the hydrologic cycle is based on a water mass balance. Soil water content is computed using the water balance equation [21]:

\[ SW_i = SW + \sum_{i=1}^{t} (R_i - Q_i - ET_i - P_i - QR_i) \]

Where, \( SW \) is the soil water content; \( i \) is time in days for the simulation period \( t \); \( R \) is the daily precipitation; and \( Q \), \( ET \), \( P \) and \( QR \) respectively, are runoff, evapotranspiration, percolation and return flow.

Other than the topographic, soil and LULC data, SWAT requires spatially explicit datasets of climatic data at daily/sub-daily time steps. Major input data for SWAT include DEM, LULC, soil properties, and daily weather data (precipitation, maximum and minimum air temperature, relative humidity, wind speed and solar radiation) [22].

3.5. SWAT - CUP

SWAT-CUP is a computer program that was developed for SWAT models. SWAT-CUP was employed for model calibration, validation, and sensitivity analysis by using the observed runoff data. The program links four calibration methods such as Generalized Likelihood Uncertainty Estimation (GLUE), Sequential Uncertainty Fitting Procedure Version 2 (SUFI2), Markov Chain Monte Carlo (MCMC) and Parameter Solution (ParaSol) [23]. SUFI2 method was chosen because it is the most suitable way to find the SWAT uncertainty under the condition that the parameter range was specified. Sequential Uncertainty Fitting Algorithm (SUFI-2) is very advantageous since it combines optimization with uncertainty analysis and can handle large number of parameter to achieve good prediction uncertainty ranges for the period of 6 years (2002 to 2007). Responded to parameter set more sensitive than any others [23, 24].

3.6. Model Performance Evaluation Procedure

There are many kinds of error parameters which are widely used for testing and accuracy assessment of the SWAT model; such as Nash-Sutcliff coefficient of efficiency (NSE), percent bias (PBIAS), coefficient of determination (R²) and RSR (ratio of the root mean square error to the standard deviation of measured data). These parameters indicate the
goodness of fit of the observed value with the simulated value. The model calibration was aimed to achieve a satisfactory model efficiency of NSE ≥ 0.5, PBIAS ≤ ± 25%, and RSR ≤ 0.7.

The determination coefficient, R² is used to determine the agreement between the simulated and observed flow data. Model prediction evaluation was categorized as satisfactory if R² is greater than 0.6 [25].

4. Results and Discussion

4.1. Selection of Suitable Climate Models and Scenarios

Ten global climate models such as CanESM2, CCSM4, CMCC-CMS, GFDL-CM3, GFDL-ESM2G, MIROC ESM, MIROC ESM CHEM, MPI ESM LR, MPI ESM MR, MRJ- CGCM3 for the 5th Coupled Model Intercomparison Project (CMIP5) are used to consider climate projections.

Here, the multi-GCMs approach is applied rather than using single GCM because different GCMs have different grid-sizes and their coverage area. R² and RMSE (root mean square error) parameters are used for the performance of the bias correction method in model selection. GCM with higher agreement on performance indicators is selected. Suitable climate models for precipitation and temperature for selected stations within the basin are shown in Table 1.

<table>
<thead>
<tr>
<th>Stations</th>
<th>Maximum Temperature</th>
<th>Minimum Temperature</th>
<th>Rainfall</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hsipaw</td>
<td>GFDL CM3</td>
<td>MRI CGCM3</td>
<td>CanESM2</td>
</tr>
<tr>
<td>Katha</td>
<td>GFDL CM3</td>
<td>MRI CGCM3</td>
<td>CCSM4</td>
</tr>
<tr>
<td>Kyaukme</td>
<td>MPI ESMMR</td>
<td>MRI CGCM3</td>
<td>CanESM2</td>
</tr>
<tr>
<td>Lashio</td>
<td>MPI ESMMR</td>
<td>MRI CGCM3</td>
<td>CanESM2</td>
</tr>
<tr>
<td>Mandalay</td>
<td>MPI ESMMR</td>
<td>MRI CGCM3</td>
<td>CanESM2</td>
</tr>
<tr>
<td>Meikhtila</td>
<td>GFDL CM3</td>
<td>MRI CGCM3</td>
<td>MIROC-ESM-CHEM</td>
</tr>
<tr>
<td>Moegaung</td>
<td>GFDL CM3</td>
<td>MPI ESMMR</td>
<td>CCSM4</td>
</tr>
<tr>
<td>Moegok</td>
<td>MRI CGCM3</td>
<td>MRI CGCM3</td>
<td>CanESM2</td>
</tr>
<tr>
<td>Myitkyina</td>
<td>MRI CGCM3</td>
<td>MRI CGCM3</td>
<td>GFDL CM3</td>
</tr>
<tr>
<td>PutaO</td>
<td>MRI CGCM3</td>
<td>MRI CGCM3</td>
<td>GFDL-ESM2G</td>
</tr>
<tr>
<td>Sagaing</td>
<td>GFDL CM3</td>
<td>MRI CGCM3</td>
<td>CanESM2</td>
</tr>
<tr>
<td>Shwebo</td>
<td>MPI ESMMR</td>
<td>MRI CGCM3</td>
<td>CanESM2</td>
</tr>
<tr>
<td>Yamethin</td>
<td>GFDL CM3</td>
<td>MRI CGCM3</td>
<td>CanESM2</td>
</tr>
<tr>
<td>YeU</td>
<td>MPI ESMMR</td>
<td>MRI CGCM3</td>
<td>CanESM2</td>
</tr>
</tbody>
</table>

4.2. Projected Precipitation

Future climate projection baselines (2021-2095) were compared to the meteorological data (1991-2015). Precipitation projections are performed under three horizons: near future for 2021-2045, future for 2046-2070, far future for 2071-2095. Projections of precipitation under RCP2.6, RCP4.5, and RCP8.5 are shown in the following Table 2. This table shows the values of average seasonal changes in precipitation as a fraction of the base period corresponding to scenarios. The seasonal climate in Upper Ayeyarwaddy River Basin is classified into three seasons such as summer from 16 Feb to 31 May, Rainy (Monsoon) from 1 June to 30 Sept and winter from 1 Oct to 15 Feb. RCP2.6 shows an increase in basin average annual precipitation by 20.2, 25.7, and 22.1% respectively for the near future, future, and far future. RCP 8.5 has a slightly higher increase rate of 27.1, 18.5, and 29.1% for the same periods. The highest increase of 29.1% and the lowest increase of 18.7% will be obtained respectively under RCP8.5 and RCP4.5 in the far future. According to the average seasonal changes, there is a clear estimation in predicted future precipitation amounts of RCP 2.6, RCP4.5 and RCP8.5 receiving reduced rainfall for all future periods in summer season and increased rainfall in rainy and winter season. But, seasonal changes in winter are predicted to become double in the future and it can be said that changes in future precipitation for the winter season are much higher than that of changes in the rainy season.
Table 2. Average Changes in Future Precipitation

<table>
<thead>
<tr>
<th>Future Period</th>
<th>Average Seasonal and Annual Changes in Precipitation as a Fraction of Base Period Corresponding to Scenarios</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RCP2.6</td>
</tr>
<tr>
<td></td>
<td>Rainy Seasonal Changes</td>
</tr>
<tr>
<td>Near Future</td>
<td>1.30 (30.5%)</td>
</tr>
<tr>
<td>Future</td>
<td>1.29 (29.3%)</td>
</tr>
<tr>
<td>Far Future</td>
<td>1.20 (20.5%)</td>
</tr>
<tr>
<td></td>
<td>Summer Seasonal Changes</td>
</tr>
<tr>
<td>Near Future</td>
<td>0.59 (-40.3%)</td>
</tr>
<tr>
<td>Future</td>
<td>0.72 (-28.1%)</td>
</tr>
<tr>
<td>Far Future</td>
<td>0.82 (-18.2%)</td>
</tr>
<tr>
<td></td>
<td>Winter Seasonal Changes</td>
</tr>
<tr>
<td>Near Future</td>
<td>2.17 (116.9%)</td>
</tr>
<tr>
<td>Future</td>
<td>1.77 (76.9%)</td>
</tr>
<tr>
<td>Far Future</td>
<td>1.88 (88.3%)</td>
</tr>
<tr>
<td></td>
<td>Average Annual Changes</td>
</tr>
<tr>
<td>Near Future</td>
<td>1.20 (20.2%)</td>
</tr>
<tr>
<td>Future</td>
<td>1.26 (25.7%)</td>
</tr>
<tr>
<td>Far Future</td>
<td>1.22 (22.1%)</td>
</tr>
</tbody>
</table>

4.3. Projected Temperature

The future projection of average temperature changes on fourteen stations under climate scenarios, which are RCP2.6, RCP4.5 and RCP8.5, are analysed in there periods, near future for 2021-2045, future for 2046-2070, far future for 2071-2095. According to the future seasonal temperature changes, both maximum and minimum temperature is projected to increase in all scenarios for three future periods and shown in Table 3 and Table 4. However, the slight increment is occurred under RCP2.6 and ranged from 0.19°C to 1.57°C for maximum seasonal temperature. Under RCP4.5, maximum seasonal temperature changes increased with the range from 0.53°C to 2.28°C and the highest increment occurs between 0.55°C to 3.76°C under RCP8.5. However, the slight increment is occurred under RCP2.6 and ranged from 0.19°C to 1.57°C for maximum seasonal temperature. Under RCP4.5, maximum seasonal temperature changes increased with the range from 0.53°C to 2.28°C and the highest increment has occurred between 0.55°C to 3.76°C under RCP8.5. The seasonal increases in the average minimum temperature range from 0.78°C to 1.48°C under RCP2.6, 0.98°C to 2.25°C under RCP4.5 and 1.12°C to 3.92°C under RCP8.5.

Table 3. Average Seasonal and Annual Changes in Future Maximum Temperature

<table>
<thead>
<tr>
<th>Future Period</th>
<th>Average Maximum Temperature Changes Based on Base Period Corresponding to Scenario (°C)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RCP2.6</td>
</tr>
<tr>
<td></td>
<td>Rainy Seasonal Changes</td>
</tr>
<tr>
<td>Near Future</td>
<td>0.84</td>
</tr>
<tr>
<td>Future</td>
<td>0.93</td>
</tr>
<tr>
<td>Far Future</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td>Summer Seasonal Changes</td>
</tr>
<tr>
<td>Near Future</td>
<td>1.25</td>
</tr>
<tr>
<td>Future</td>
<td>1.45</td>
</tr>
<tr>
<td>Far Future</td>
<td>1.57</td>
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<td></td>
<td>Winter Seasonal Changes</td>
</tr>
<tr>
<td>Near Future</td>
<td>0.19</td>
</tr>
<tr>
<td>Future</td>
<td>0.5</td>
</tr>
<tr>
<td>Far Future</td>
<td>0.76</td>
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<td></td>
<td>Average Annual Changes</td>
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<td>Near Future</td>
<td>0.59</td>
</tr>
<tr>
<td>Future</td>
<td>0.81</td>
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<tr>
<td>Far Future</td>
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</table>
Table 4. Average Seasonal and Annual Changes in Future Minimum Temperature

<table>
<thead>
<tr>
<th>Future Period</th>
<th>Average Minimum Temperature Changes Based on Base Period Corresponding to Scenario (˚C)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RCP2.6</td>
</tr>
<tr>
<td>Rainy Seasonal Changes</td>
<td></td>
</tr>
<tr>
<td>Near Future</td>
<td>1.05</td>
</tr>
<tr>
<td>Future</td>
<td>1.28</td>
</tr>
<tr>
<td>Far Future</td>
<td>1.43</td>
</tr>
<tr>
<td>Summer Seasonal Changes</td>
<td></td>
</tr>
<tr>
<td>Near Future</td>
<td>1.13</td>
</tr>
<tr>
<td>Future</td>
<td>1.24</td>
</tr>
<tr>
<td>Far Future</td>
<td>1.48</td>
</tr>
<tr>
<td>Winter Seasonal Changes</td>
<td></td>
</tr>
<tr>
<td>Near Future</td>
<td>0.78</td>
</tr>
<tr>
<td>Future</td>
<td>1</td>
</tr>
<tr>
<td>Far Future</td>
<td>1.26</td>
</tr>
<tr>
<td>Average Annual Changes</td>
<td></td>
</tr>
<tr>
<td>Near Future</td>
<td>0.98</td>
</tr>
<tr>
<td>Future</td>
<td>1.16</td>
</tr>
<tr>
<td>Far Future</td>
<td>1.39</td>
</tr>
</tbody>
</table>

4.4. Soil and Water Assessment Tool (SWAT) Model Data Requirements and Set-up

SWAT is a partially distributed model and required digital elevation model (DEM), land use and soil map which are basic modeling requirements and daily weather data. There are many available global sources for DEM data and the 90m resolution DEM derived from SRTM (NASA Shuttle Rader Topographic Mission) is used to set up for the hydrological modeling by SWAT. The DEM in Figure 3 shows that the elevation in the Upper Ayeyarwady river basin ranges from 50m in the lower plains to 5711m in the upper region. The DEM is an important parameter for SWAT to classified topography, landscape, elevation, and slope and used in Streamflow network construction for modelling simulation. In SWAT, the first process was automatic watershed delineation and it can define the detail of flow direction and accumulation of each and every part of the large watershed which was divided into 23 sub-watersheds. These sub-basins were then divided into Hydrologic Response Units (HRUs) to predict runoff separately.
For the rainfall-runoff relationship, landcover and soil digital map are used to simulate the model. Land cover in Figure 4 has a 90m grid resolution with a 15 class landcover classification scheme and is obtained from the Servir Mekong Land Cover Portal. Soil dataset with 90m grid resolution is generally classified into 17 classes in Figure 5. For this study, thresholds for defining HRUs to compute a water balance based on snow, soil, shallow aquifer and deep aquifer were set at 20% for soil and 10% for land use [27]. Land cover map and soil map were prepared with their lookup tables to join raster data in the SWAT database file. SWAT model requires a soil map and a database table of soil texture for available water content, hydraulic conductivity, bulk density and organic carbon content for different layers of each soil type. The main hydrological processes include infiltration, runoff, evapotranspiration, lateral flow, and percolation [28]. Precipitation, maximum and minimum temperature, relative humidity, sunshine hours and wind speed of 1991 to 2013 were prepared in the text file for each station to compute weather generator parameters for the simulation of hydrological process on the basin. After that, different methods of water balance, surface runoff and reaches are defined.

![Figure 4. Landcover Classification Map](image1)

![Figure 5. Soil Classification Map](image2)
Surface runoff can be simulated by two methods in the SWAT model: the modified Soil conservation Service Curve Number Method and the Green-Ampt Infiltration Method. In this study, surface runoff was estimated using the SCS Curve Number. Among three methods for estimating potential evapotranspiration: Priestley & Taylor, the Panmen-Monteith and Hargreaves & Samani, Panmen-Monteith method is used for calculation of evapotranspiration and soil and snow evaporation. Variable Storage routing was used for channel routing [29].

4.5. Calibration and Validation of Streamflow

Daily river flow (m³/sec) observed at the Sagaing gauging station which is the outlet of the Upper Ayeyarwady river basin was used for the model calibration and validation analysis. Generally, hydrological models require a “warm-up” period, defined as the time the model will run before starting to generate the actual outputs, in order to eliminate the initial bias. In this study, the period 2000-2001 was used as “warm-up” periods to allow the model to initiate the hydrological parameters. The period of 2002 to 2007 of the streamflow data was used for calibration to estimate the model parameters values and the stability of these parameters were tested in the validation period of 2008 to 2013. Calibration was performed using the SWAT CUP program. Among the four calibration methods, calibration, and validation were conducted using the Sequential Uncertainty Fitting Algorithm (SUFI-2). Several hydrological model parameters were adjusted for achieving the best fit between the simulated and measured flow at the monitoring station. The total of twenty parameters as relating to surface hydrology, groundwater hydrology, snowpack accumulation, snowmelt, and base flow was selected and the lists of initial and fitted parameter range over all the study basins are reported in Table 5. Both calibration and validation on simulated streamflow with observed data were done by using the final parameters ranges.

<table>
<thead>
<tr>
<th>Table 5. Parameters for Sensitivity Analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td>Parameters</td>
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<tr>
<td></td>
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<tr>
<td>------------</td>
</tr>
<tr>
<td>Groundwater Parameters</td>
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<tr>
<td>GW_DELAY.gw</td>
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<td>ALPHA_BF.gw</td>
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<td>GWQMN.gw</td>
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<tr>
<td>GW_REVAP.gw</td>
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<tr>
<td>RCHRG_DP.gw</td>
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<tr>
<td>Surface Parameters</td>
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</tr>
<tr>
<td>SURLAG.bsn</td>
</tr>
<tr>
<td>OV_N.rte</td>
</tr>
<tr>
<td>CH_N2.rte</td>
</tr>
<tr>
<td>CH_K2.rte</td>
</tr>
<tr>
<td>HRU_SLP.hru</td>
</tr>
<tr>
<td>SLSUBBSN.hru</td>
</tr>
<tr>
<td>Soil Parameters</td>
</tr>
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</tr>
<tr>
<td>SOL_K.sol</td>
</tr>
<tr>
<td>SOL_BD.sol</td>
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<td>ESCO.hru</td>
</tr>
<tr>
<td>EPCO.hru</td>
</tr>
<tr>
<td>Snow Parameters</td>
</tr>
<tr>
<td>SFTMP.bsn</td>
</tr>
<tr>
<td>SMTMP.bsn</td>
</tr>
</tbody>
</table>

1) Statistical Approaches for model performance evaluation: For the accuracy of simulated model results, the model calibration has to be done to match modelled streamflow results with observed discharge. Surface runoff was calibrated many times by adjusting the parameters to compare with observed data. In this study, the model performance was checked with Nash-Sutcliffe efficiency (NSE), coefficient of determination (R²), percent bias (PBIAS), and ratio of the root mean square error to the standard deviation of measured data (RSR). The calibration process was done until NSE >
0.5, $R^2 > 0.5$, $RSR \leq 0.7$, $PBIAS < 25\%$. The validation process is also performed to check the calibrated model accuracy. The statistical result of model validation indicates that the model can be used with calibrated basin parameters to simulate flow for the future period with considerable reliability and the model can give reliable results with projected discharge data for the future period. Model fitness between the daily observed and simulated runoff during the calibration and validation period is presented in Table 6.

<table>
<thead>
<tr>
<th>Period</th>
<th>$R^2$</th>
<th>NSE</th>
<th>PBIAS</th>
<th>RSR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calibration (2002-2007)</td>
<td>0.88</td>
<td>0.88</td>
<td>22.1</td>
<td>0.35</td>
</tr>
<tr>
<td>Validation (2008-2013)</td>
<td>0.88</td>
<td>0.87</td>
<td>22.8</td>
<td>0.36</td>
</tr>
</tbody>
</table>

2) Monthly Time Series Simulation for Model Calibration: When the $R^2$ and NSE values improved to 0.88 and 0.88 while decreasing the value of PBIAS and RSR to 22.1 and 0.35 respectively, simulation is seen to be better. According to Figure 6, the time of maximum rainfall intensity was also corresponded to the time of peak flow during the year.

![Figure 6. Hydrograph of Monthly Simulated and Observed Flow for Calibration Period (2002-2007, Sagaing Outlet)](image)

3) Monthly Time Series Simulation for Model Validation: The values of statistical indicators: $R^2$, NSE, PBIAS, and RSR are all satisfied with the value of 0.88, 0.87, 22.8 and 0.36. The hydrograph of observed and simulated discharge for the validation period is shown in Figure 7 and better model performance results were attained during monthly time series calibration periods.

![Figure 7. Hydrograph of Monthly Simulated and Observed Flow for Validation Period (2008-2013, Sagaing Outlet)](image)
All the statistical parameters are satisfactory at both calibration and validation period. $R^2$ with the value of 0.88 also indicates good agreement of the simulated flows with the observed flows during calibration and validation period. NSE values are 0.88 and 0.87 for the calibration and validation period, respectively. The monthly PBIAS values are within the limitations indicating in the value of 22.1% in calibration and 22.8% in validation period. RSR also reduces to acceptable limit and all these values indicate that the simulated and observed discharge has a reasonable agreement and the model performance is acceptable.

4.6. Changes in Average Annual and Seasonal Streamflow at Sagaing Station

The same values of calibrated SWAT model parameters are applied to evaluate the future climate impact simulations on water resources analysis. The bias-corrected future climate parameters such as precipitation, maximum and minimum temperature obtained from the selection suitable climate models for each station in the study area are used as input climate data for the analysis of hydrological changes under future climate. To evaluate the impact of climate change on the hydrology of the Upper Ayeyarwady river basin, future hydrological projections divided into three-time horizons Near Future (2021-2045), Future (2046-2070) and Far Future (2071-2095) were compared with their baseline average discharge (1991-2015). And then, analysis of future streamflow at Sagaing Outlet is performed under RCP2.6, RCP4.5, and RCP8.5 climate scenarios. The results of average annual and seasonal change (%) in the basin relative to the base period are shown in Figure 8. According to this figure, seasonal streamflow shows a decreasing trend in all scenarios and periods of the near future, future and far future. But the decrease rate in summer is higher than in the other two seasons. The future average seasonal streamflow shows a definite decrease ranging from 12.9% to 84.5% will be lower than the current conditions in the summer season. Average seasonal streamflow will also decrease ranging from 2.8% to 15.9% in the rainy season and from 10.6% to 14.6% in the winter season. A slight increment of about 13.6% occurs under RCP2.6 in the far future for winter. There is still clear agreement that the projected annual streamflow during near future, future and far future will be 6.1% to 30.9% lower than observed annual streamflow amounts as compared to the observed annual streamflow. About 1.3% increment occurred under RCP2.6 in the far future of annual streamflow projection.

Figure 8. Future Average Annual and Seasonal Streamflow Changes at Sagaing Station

4.7. Changes in Monthly Streamflow at Sagaing Station

The comparison between observed and projected average monthly streamflow at the Upper Ayeyarwady river basin for each future period is described in the following Figures. Figure 9 shows the projection of average monthly streamflow for the near future period under three RCP scenarios and figure 10 shows near future average monthly streamflow changes relative to the baseline period. In these figures, the average monthly streamflow is higher than the baseline period from January to May for all RCP scenarios, June for RCP8.5, and October to December for RCP 4.5 and RCP 8.5. In near future, slight increase streamflow is observed under RCP 4.5 in June, July, August, and September, under RCP2.6 in June and September, and under RCP8.5 in August and September. For the summer season (15February to 31May) for near future, streamflow projection under all RCP scenarios indicates a decrease projection change in average of 1932.1 m$^3$/s under RCP2.6, 1465.11 m$^3$/s under RCP4.5, and 845.9 m$^3$/s under RCP 8.5. The average change in streamflow in summer is at the lowest level under RCP8.5 compared with other RCP scenarios. During the rainy season, the highest and lowest streamflow changes are ranging from about 10106.6 – 16441.5 m$^3$/s under RCP 2.6, 9415.6 - 15324 m$^3$/s under RCP4.5, and 9252.4 - 16908 m$^3$/s under RCP8.5 in near future.
In the future, the average monthly streamflow is higher than the baseline period from January to May for all RCP scenarios, June for RCP 8.5, and October to December for RCP 4.5 and RCP 8.5. Simulated streamflow higher than observed streamflow is given for three months: July, August, and September under all RCPs and for June under RCP2.6 and RCP4.5. The summer season is also affected by average decreasing changes in future of 1530.6 m$^3$/s under RCP 2.6, 1213.1 m$^3$/s under RCP 4.5, and 724 m$^3$/s under RCP 8.5. The rainy seasonal flow projections under RCP2.6, RCP4.5 and RCP8.5 conditions for future are also ranging from about 10060 – 16333.9 m$^3$/s, 10680 – 16889.4 m$^3$/s, and 9701.3 - 17566 m$^3$/s, respectively.

The projection of average monthly streamflow with respect to the baseline under all RCPs for the future is shown in Figure 11. This figure shows the declining streamflow changes during the summer and winter season (from January to May and from October to December) under all scenarios. In July and August, for the far future period, the simulated flow of RCP8.5 is higher than the flow of RCP 2.6 and RCP4.5 after comparing it with the baseline period. Average decrease changes of streamflow projection for summer season in far future is about 992.9 m$^3$/s under RCP 2.6, 751.1 m$^3$/s under RCP 4.5, and 448.7 m$^3$/s under RCP 8.5. The rainy seasonal flow projections are also ranging from 10159.5 - 16368.4 m$^3$/s under RCP 2.6, 9701.3 - 17066 m$^3$/s under RCP4.5, and 8486.3 - 17918 m$^3$/s under RCP 8.5.
In terms of seasonal scale, it is clear that the projected streamflow will decrease in the summer period. Both increasing and decreasing streamflow are found in the rainy and winter period according to their RCP scenarios.

### 4.8. Impact on High and Low Flow of Upper Ayeyarwady River Basin at Sagaing Outlet

According to the future high flow results shown in Figure 12, projected future streamflow has an increasing trend from RCP2.6 to RCP8.5 from the year 2021 to 2095 indicates severe floods will ever experience in the future. It also means that water-related disasters such as floods, landslides, etc. will be more encountered in RCP4.5 and RCP8.5 conditions than in RCP2.6.

![Projected Streamflow Graph](image1.png)

**Figure 11. Projected Average Monthly Streamflow in Far Future (2071-2095)**

![Projected Maximum Discharge Graph](image2.png)

**Figure 12. Projected Maximum Discharge in Sagaing Outlet for Wet Season**

![Projected Minimum Discharge Graph](image3.png)

**Figure 13. Projected Minimum Discharge in Sagaing Outlet for Dry Season**

Figure 13 describes the projected future low flow from the year 2021 to 2095 and it is forecasted as the streamflow will decrease more and more in relation to their future scenarios and that indicates water problem will grow worse in the future.
The relative changes of high and low flow of Upper Ayeyarwady River Basin concerning the baseline period (1991-2015) are presented in Figure 14. The projected high flow indicated that the flow will be increased ranging from 3.1% to 12.7% under RCP2.6, 2.8% to 33.5% under RCP4.5, and 25.7% to 28.9% under RCP8.5. The range of changes in low flow is 5.1% to 13% under RCP2.6, 37.1% to 48.3% under RCP4.5, and 64.2% to 74.2% under RCP8.5. Here, changes percentage in low flow is greater than that of high flow. Both increasing rates in high flow and decreasing rate in low flow are more serious under RCP8.5 than RCP2.6 and RCP4.5.

Figure 14. Projected Changes Percentage in High and Low Flow of Upper Ayeyarwady River Basin

5. Conclusion

The assessment of climate change impacts on hydrology is related to the climate change scenarios and hydrological model. Runoff projection with GCMs, emission scenarios and future periods is investigated as well. Changes of Upper Ayeyarwady river flow during 2020-2100 were predicted by using projected precipitation and temperature related to outputs of selected best climate models form 10 GCMs under RCP2.6, RCP4.5, and RCP8.5 scenarios. Estimating the impacts of possible future climate change on hydrological behaviour is done by calibration and validation of the SWAT hydrological model by using current climatic inputs, land-use map, and observed river flow. Better model performance results were attained a satisfactory value of $R^2 = 0.88$, NSE = 0.88, PBIAS = 22.1 and RSR = 0.35 during monthly time series calibration period and value of $R^2 = 0.88$, NSE = 0.87, PBIAS = 22.8 and RSR = 0.36 during validation period. The overall conclusion of this research is that the Upper Ayeyarwady area predicted to encounter excessive precipitation especially in the rainy season and extreme temperature especially in the future summer season. And then, it can also be concluded that river flow followed the rainfall pattern because the river flow is found as the high flow in the dry season and high flow in the wet season. Moreover, changing climate conditions are getting worse with the increased rate of GHG emissions levels in the atmosphere and increasing in frequency and intensity of rainfall lead to floods and increasing in water scarcity and drought. It is found that current Myanmar’s climate condition is still below the RCP2.6 level according to the comparison between observed and simulated data and therefore, it is necessary to control and reduce flood potentials in the wet season and the severity of drought in the dry season. But, if the greenhouse gas emissions rate in Myanmar cannot reduce, the level of gas will keep going up and face to reach the RCP4.5 and RCP8.5 conditions. Therefore, stronger efforts and finding ways to reduce and to mitigate carbon dioxide emissions should be undertaken to lower global warming and climate change in the atmosphere.

6. Acknowledgements

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7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


Environmental and Economic Analysis of Selected Pavement Preservation Treatments

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Abstract

Pavements are one of the highest assets and represent massive investment. The need to design and provide a sustainable maintenance service is becoming a priority and this comes mutually with the intentions to reduce impacts caused by maintenance treatments to the environment. This paper through a case study presents a Life Cycle Cost and Assessment technique during a 30 year analysis period to measure the cost effectiveness, embodied energy and carbon emissions of selected preservation treatments. These treatments can either be applied separately or in combination during the preventive maintenance of road pavements. This study entails three life cycle phases of material extraction and production, transportation and construction of maintenance activities. Through a literature review, raw materials energy and emission inventory data was averaged followed by the analysis of the equipment involved by using the specific fuel consumption to calculate the energy and emissions spent by the machine and finally the selected treatment energy and emissions was computed. Results show that preservation treatments can have an LCC of 30-40% and embodied energy and carbon emission of 3-6 times lower than the traditional approach. This study bridges gaps in literature on integrated evaluation of environmental and economic aspects of preservation treatments.

Keywords: Pavements; Sustainability; Life Cycle Assessment; Life Cycle Cost; Preventive Maintenance.

1. Introduction

The road network has an important role to play in the development of a country for social and economic growth. A good road network is also important for connectivity, movement of goods and job creation. Zambia has a total gazette road network of 67671 km of which 60% comprises the Core Road Network (CRN). The CRN infrastructure in Zambia consists of a sparsely interconnected network of Trunk (T), Main (M), District (D), Primary Feeder (PF) and Urban (U) roads [1].

The Government of the Republic of Zambia (GRZ) has implemented three notable initiatives in the road sector. Firstly, to improve inter-urban and urban connectivity and accessibility; this will see over 12000km of roads rehabilitated or upgraded to bituminous standard at a total cost of US$8.5 billion. Secondly, GRZ has developed a ten year (2015 – 2024) National Maintenance Strategy which aims to reduce road maintenance backlog and to improve the general condition of CRN. The estimated cost of implementing the entire strategy over the 10 years is US$1.5 billion [2]. Thirdly, the Output and Performance Based Road Contracting (OPRC), which underpins sustainability in road maintenance, is a major key decision adopted [2-3].

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The pre-occupation over the years has mostly been with new constructions and upgrading at the expense of road maintenance. The rehabilitation technique most currently used for asphalt pavements on highways is an asphalt overlay. Pavements in Zambia have structurally deteriorated to a great degree because implementation of the pavement maintenance strategy has not yet been fulfilled. It is clear that with such a large pavement network, the GRZ is challenged to maximize available funds to maintain the network in the best condition possible [2].

It is clear that under the current policies and funding levels in the road sector, it would be inevitable to expect further deterioration in the quality of pavements. It is therefore wise to start implementing cost effective methods of preserving the existing pavements. Various studies have shown that waiting for the pavement to deteriorate beyond its service life before attempting to repair is tantamount to having major rehabilitation and reconstruction activities which would come with huge sums of money that a developing country like Zambia does not need at the moment [4-8]. In Figure 1 below, Galehouse et al. estimated that $1 invested towards pavement preservation at the beginning of the pavement life cycle could save in excess of $5 in the future [7].

![Figure 1. Relationship between pavement condition and life cycle costs](image)

With the impeding energy crisis and climate change impacts the world is facing, reducing the environmental impacts of pavement preservation treatments has become mandatory. Most published papers on environmental LCA have been conducted in Europe, USA, China but limited work has been done in Africa. The predominance of these studies lacks an integrated evaluation of both environmental and economic aspects of pavement preservation treatments [9-11]. The few studies which have conducted such analyses show variations in the base data, goal and scope definitions and analysis methods and thus comparison among such processes cannot be readily made [12-15]. It is for this reason that a life cycle assessment of cost, embodied energy, carbon emission (CO$_2$e) has been developed encompassing eleven treatment scenarios suiting the Zambian landscape. This LCA model not only considers the material extraction and production, construction maintenance but also the disposal into its system boundary.

The main objective of this study is to conduct; i) an LCCA by finding the cost effectiveness of selected preservation treatments (Micro-surfacing, Chip+Fog Seal, Thin Hot Mix Asphalt (HMA) Overlay, Mill & Fill) to be applied on Zambian roads by identifying and evaluating the cost benefits of the preservation treatments compared to the “do nothing” alternative; ii) an LCA of maintenance activities by comparing the embodied energy consumptions and carbon emissions of the studied treatments.

The goal of this paper is to quantify and compare the life cycle environmental and economic performance of multiple maintenance preservation treatments in order to improve the pavement sustainability in Zambia. The scope follows a cradle to grave approach and takes into account guidelines outlined by International Organization for Standardization 14044.

### 2. Literature Review on Preservation Treatments, Life Cycle Cost and Assessment Techniques

#### 2.1. Preservation Treatments

Pavements are one of the highest assets and represent tremendous investment. With the vast amount of resources dedicated to pavements, topped with the fact that they are under constant public scrutiny, it is imperative that the serviceability of pavements be maintained in an efficient and effective manner to get the most out of investments [16]. The most effective method for maintaining pavement serviceability is to implement a preservation program, which is a planned system of pavement surface treatments designed to extend the life of a pavement using the fewest possible resources (money, materials, energy and time). To sum up the objective of pavement preservation program: it is deciding on “the right treatment on the right pavement at the right time” [16-17].
Preventive Maintenance, as defined by American Association of State Highway and Transportation Officials (AASHTO), is “a planned strategy of cost effective treatments to an existing roadway system and its appurtenances that preserve the system, retards future deterioration and maintains or improves the functional condition of the system without substantially increasing structural capacity” [18]. Preventive maintenance is a tool utilized for pavement preservation. It is specifically utilized on pavements that are in good condition with a considerably long service life. There are three main components of pavement maintenance: routine maintenance, preventive maintenance and minor rehabilitation [18]. The pavement maintenance, no matter the choice amongst the three, is to extend the service life of a road, minimize the life cycle cost and reduce the environmental impacts. What is worth noting here is that, these treatments are applied on pavements with a high structural support capacity and having minimal distresses [18]. This paper examines and briefly explains the four types for use on Zambian roads.

2.2.1. Micro-Surfacing

A mixture of cationic polymer modified asphalt emulsion, mineral aggregate, mineral filler, water and other additives properly and carefully proportioned, mixed and laid on a paved surface. Micro-surfacing is much more effective than chip and slurry seals. It differs from chip and slurry seals in that the latter uses a thermal curing process while micro-surfacing uses a chemically controlled process. Owing to its robustness and flexibility, micro-surfacing can be used as a preventive, routine and corrective maintenance strategy [19-21].

2.2.2. Chip Seal + Fog Seal

This is an application of asphalt binder on existing pavement followed by a layer of aggregate chips. The treatment is then rolled to embed the aggregate into the binder. There are a number of variations of chip seals, the more common ones being single and double chip seals. A single chip seal is an application of binder followed by the aggregate while a double chip seal is a built up seal coat consisting of multiple applications of binder and aggregate. To put it into perspective, a double chip seal involves a spray application of binder, spreading a layer of aggregate, rolling the aggregate for embedment, then applying an additional application of binder, spreading another layer of aggregate usually half the base aggregate gradation and finally rolling [22].

Fog Seal is an application of diluted emulsion to augment the pavement surface and retards ravelling and oxidation. It involves the application of a slow setting asphalt emulsion diluted with water normally in the ratio of 1 to 1 sprayed directly to an existing pavement. The sole purpose is to renew the old HMA that has become brittle and dry. This treatment has been found to not only seal surface voids and cracks but also to inhibit ravelling [23].

2.2.3. Thin Hot Mix Asphalt Surfacing

This treatment is used for maintenance and/or rehabilitation, a method used to extend the life of pavements that are structurally sound and have minimal rutting. A thin overlay consists of an overlay of hot mix asphalt of 30-40mm thickness. It is a mixture of well graded aggregates, asphalt binder and filler mixed hot in a mixing plant. The types of hot mix asphalt surfacing are Dense-Graded Systems, Open-Graded Systems, Gap-Graded Systems and Ultra-Thin Systems (NovaChip®). This preservation technique requires careful attention to the material mix design procedure. Things to consider include high quality aggregates, the use of a softer asphalt binder or modified binders and the mix design itself should be similar to HMA structural mixes [24-25].

2.2.4. Mill and Fill

This is a better preventive maintenance strategy compared to simple HMA overlay, in which 20-25mm of the old asphalt is milled out and then resurfaced with a 40-50mm of new HMA. Milling is not expensive and has low emissions, but is nonetheless more beneficial to a pavement with irregularities. The advantages of milling include; compatibility of the new pavement with the existing drainage system, maintaining the required clearance for vertical obstacles and protection of new layers from the propagation of the distress existing in the previous layers. The final pavement of a mill and resurface has the same effect as a thin overlay with an added advantage of preventing the accumulation of asphalt layers thereby improving the vertical load transmission to the deepest layers [25-26].

2.2. Life Cycle Cost Analysis (LCCA)

This is a decision support tool often used by road agencies to compare total user and agency cost for different treatment alternatives. It is an economic analytical tool that compares benefits and costs of the selected alternatives and allows decision makers to choose the best option. There are basically two types of approaches that could be employed in LCCA: deterministic and probabilistic. In the deterministic approach, input variables are considered discrete fixed variables. The biggest drawback of this traditional approach is that it does not account for the variability associated with the LCCA input parameters. This level of uncertainty is mainly a combination of four reasons as highlighted by a study done by Tighe [27-28]. These uncertainties can be combatted using the probabilistic approach or a sensitivity analysis. The use of computer simulation software (RealCost) is the most utilized method for probabilistic approach [28].

Vol. 6, No. 2, February, 2020

212
LCCA is a method based on the principles of economics to evaluate the long term economic benefits of the different investment options. This method is been utilized by a large number of agencies worldwide due to its ability to analyze pavement economics realistically [29-31]. The Federal Highway Administration (FHWA) technical bulletin lists the steps involved in conducting a life cycle cost analysis [29].

The cost components for use in the analysis are initial costs, maintenance cost, rehabilitation, user costs and salvage value. The equations to be used for the economic analysis will be the Net Present Value (NPV) and the Equivalent Uniform Annual Cost (EUAC). The reason for choosing these two equations is the fact that they are the most widely used economic indices available worldwide [27, 32-33].

Equation 1 shows the NPV formula.

\[
NPV = Initial\ Cost + \sum_{k=1}^{N} Future\ Cost_k \left[ \frac{1}{(1+i)^n_k} \right] - Salvage\ Value \left[ \frac{1}{(1+i)^n_e} \right]
\]

Where:
- \( N \) = number of future costs incurred over the analysis period,
- \( i \) = discount rate as a percentage,
- \( n_k \) = number of years from the initial construction to the \( K \)th expenditure,
- \( n_e \) = analysis period in years.

Since the Zambian budget is presented annually, it is imperative that our cost matches the budget timing. It is for this reason that the present and future expenditures are converted to a uniform annual cost in the form of EUAC. Equation (2) shows the EUAC formula.

\[
EUAC = NPV \left[ \frac{(1+i)^n}{(1+i)^n-1} \right]
\]

Where:
- \( NPV \) = Net Present Value
- \( i \) = discount rate,
- \( n \) = years of expenditure.

Finally, after the computations of the LCCA of pavements, the present values of the differential costs are compared across competing alternatives.

### 2.3. Life Cycle Assessment (LCCA)

Life Cycle Assessment (LCA), as defined by the International Organization for Standardization (ISO) 14040 and 14044, is a tool that makes it possible to assess the environmental impact of a product. It is a process or an activity, through identifying and quantifying the flows of energy and materials, evaluating the consumption of energy and materials as well as emissions generated, and identifying and evaluating possible measures for improving the environment [34-36].

This LCA approach formalized by ISO 14040 series divides the LCA framework into four interactive stages: i) goal and scope definition; ii) life cycle inventory analysis (LCI); iii) life cycle impact assessment (LCIA); iv) interpretation. The goal and scope outlines the reasons for conducting the study as well as the intended application and audience. In this stage the system boundaries which involve the picking of activities or processes to be incorporated in the LCA and functional unit has been defined as quantifying a stated amount of a system for use as a reference unit is well expounded. The LCI, which is the second stage, compiles the inputs and outputs from the product over its life cycle in relation to the functional unit. The LCIA aims to form a link between the system and the potential to cause human and environmental damage. Finally, the results from all the previous stages are evaluated and interpreted in relation to the goal and scope definition in order to identify analysis refinements and improvements and to reach a conclusion to recommend and aid in the decision making process [34-35]. Currently, the three methods of LCA commonly employed include: Economic Input-Economic Output (LCA-EIO), Process based LCA, and Hybrid LCA [37].

The LCA-EIO employs a top down approach to critically relate production of goods and services to the production outputs of the sectors of an economy. It does not require a system boundary but traces all direct and indirect economic inputs required to produce a unit of output from a given economic sector [37]. Horvath and Hendrickson used this method to compare the energy consumption of Hot Mix Asphalt (HMA) and Continuously Reinforced Concrete Pavement (CRCP). The study mainly dwelt on extraction and production of different surface materials and the qualitative analysis of construction phase and end of life. They concluded that the HMA consumed 40% more energy than the CRCP [38]. The process based method involves the principles refined by Society of Environmental Toxicology and Chemistry (SETAC) and U.S. Environmental Protection Agency (EPA) [36, 49]. It provides a transparent bottom up approach for assessing process based environmental contributions like carbon emissions within the defined boundary. Stripple utilized it and studied Jointed Plain Concrete Pavements (JPCP) and asphalt pavements constructed using both hot and cold production techniques. This study concluded that without the feedstock energy, JPCP consumes more energy than its
counterparts [39]. The hybrid assessment combines process based and LCA-EIO in a manner that exploits the strength and minimizes the limitations associated with the previous two methods. By using this method, the limitations and errors of using the conventional methods are reduced [37]. Park et al. used this method to study the environmental load of asphalt concrete and ready mix concrete in South Korea [40].

Important literature on pavement preservation LCA processes include a study done by Chehovits and Galehouse which studied the energy use and Greenhouse Gas (GHG) emission of preservation treatments for asphalt pavements. Common techniques like slurry seal, chip seal, Hot In Place Recycling (HIR), crack seal etc. were used. Results show that on a per annum basis, different maintenance treatments consume different amounts of energy per year of pavement life. It went on to show that new construction, thin HMA and HIR have the highest energy usage rate while slurry, chip seal and micro-surfacing have lower amounts of energy per year [11]. A study conducted in Chile about different asphalt pavement maintenance and rehabilitation techniques found out that Cold In Place Recycling (CIR) uses the least amount of energy and the haulage distance is the most sensitive factor on the total energy consumption. The different techniques included asphalt overlay, reconstruction and CIR [9]. A study by Yu and Lu compared environmental effects on three overlay systems by considering six modules; material, distribution, construction, congestion, usage and end of life. They found out that in the usage module, materials, traffic congestion and usage are the main factors for energy consumption and air emissions and that recycling materials reduces energy consumption for HMA [41].

Tatari et al. did an LCA to evaluate the environmental impacts of different types of Warm Mix Asphalt (WMA) and compared them to conventional HMA pavements. It was concluded that the WMA was less sustainable in terms of total energy [42]. A more elaborate study on asphalt concrete pavement and CRCP was studied by Hoang et al.; the energy use, emission of CO₂ and use of virgin aggregate and bitumen were the main components of this study. The results showed that CRPC consumes around 40% more energy than asphalt pavements and three times more CO₂ emissions [43]. One of the backbone literatures for LCA on pavements was studied by Chapat and Bilal; they evaluated twenty different construction techniques for calculating energy consumption and GHG emissions. The study found that heavier traffic loads require pavements of better bearing capacity and also have an increased need for maintenance operations. So it was concluded that the energy and GHG emission caused by traffic was far more than that in the construction phase [10].

3. Research Methodology

3.1. Case Study

An environmental and economic analysis was performed on an existing HMA pavement in Central Province, Zambia. The project site is the 65.5km stretch between Kabwe and Kapiri-Mposhi towns which is part of the Great North Road (T002). The existing section of the road comprises of a 3.5m single lane carriageway (both directions) and 2 m wide shoulders (inner and outer). The pavement structure consists of a 50mm asphalt concrete layer, a 150mm crushed stone base, a 150mm granular sub-base and a 300mm subgrade layer. In this case study, selected preservation treatments were evaluated over the 30 years analysis period integrating both the economic and environmental implications. The pavement design approach follows the principles contained in the 1993 AASHTO pavement design guidelines were Pavement Serviceability Index (PSI) and 80KN Equivalent Single Axle Load (ESAL) are the main parameters defining performance. These parameters are all related to the material properties, drainage and environmental conditions and performance reliability which are used to calculate the pavement structural strength through an index known as the structural number (SN). Figure 2 below shows the location map of the project.
3.2. Energy and Carbon Emission Calculation

a) Goal and Scope

The goal of this paper is to quantify and compare the life cycle environmental and economic performance of multiple maintenance preservation treatments in order to improve the pavement sustainability in Zambia. The scope follows a cradle to grave approach and takes into account guidelines outlined by ISO [34-35].

b) Functional Unit

It is defined as a 1km road of 3.5m lane width with an analysis period of 30 years.

c) System Boundaries and Assumptions

Figure 2 shows the phases and components included within the system boundaries of the proposed LCA model. The model entails three life cycle phases: materials extraction and production; transportation; and construction of maintenance treatments. The work zone traffic management and usage phase were not included in the model because of lacking a well-defined standardized method and the incognizance of the enormity of the emissions.

The construction assumptions of a maintenance project are based on case study parameters in Zambia:

- 1500 km between the refinery for bitumen production/bituminous products and mixing plant/storage place;
- 100 km between cement plant to the mixing plant/storage place;
- 50 km between aggregate quarry and mixing site/stockpile;
- 10 km between the mixing plant/stockpile/storage place and the construction site;
- 10 km between water supply and construction site;
- 20 km between construction site and the land fill.

In general, embodied energy consumption and carbon emissions occur during two main phases of pavement construction i.e., materials production and construction. The materials production phase includes the extraction and initial processing of aggregates, asphalt and other materials. The processes within this phase include raw material acquisition, transportation of raw materials to and from the plant and material manufacturing. The transportation of manufactured materials to and from the construction site is usually considered into the construction phase [10].

3.2.1. Embodied Energy and Carbon Emissions for Construction Materials

The first step in calculating the embodied energy and carbon emission of pavement preservation treatments is to determine the material components of the treatments studied. An LCI data analysis through a literature review was conducted and averages are computed in order to calculate a reasonable final value of the materials of importance as shown in Table 1 below. It should be noted that the entries listed in the table considers all stages and processes to attain the final product as ready for use.
3.2.2. Embodied Energy and Carbon Emissions for Construction Equipment

After the LCI phase through a literature review, the next step is to consider and calculate the average fuel consumption of diesel oil machines. Machinery/Equipment for the successful laying of the studied treatments like Millers, Pavers, Rollers, Chip Spreaders, Micro-surfacing Machinery, Bitumen Distributors and Trucks were investigated to identify and quantify emissions and embodied energy in road preservation treatments and activities. The primary source of emissions, in fact, is due to the engine exhaust system, depending on the total amount of fuel consumed in each phase of the preservation process. However, the true quantity of fuel consumed while applying maintenance treatment on a road is hard to estimate. The method employed in this paper is adapted from the relationship made by U.S. Environmental Protection Agency [49] to convert the calculated fuel consumption into emissions produced and energy spent. The step by step guide is outlined by Guistozzi et al. [50]. The machinery/equipment company model studied is widely used and accepted in Zambia, the outcomes calculated and analyzed from the machineries/equipments are hereby provided.

Table 2. Embodied energy and carbon emissions for construction equipment

<table>
<thead>
<tr>
<th>Model</th>
<th>Engine Rating (kW)</th>
<th>Width (m)</th>
<th>Speed (m/h)</th>
<th>Productivity (m²/h)</th>
<th>F (l/h)</th>
<th>Fₛₑ (l/m²)</th>
<th>CO₂e (g/m²)</th>
<th>Energy (MJ/m²)</th>
<th>Company</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Pavers</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>DF145C</td>
<td>153</td>
<td>5.5</td>
<td>1200</td>
<td>6600</td>
<td>38.43</td>
<td>0.0058</td>
<td>15.43</td>
<td>0.20</td>
<td>Dynapac</td>
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<tr>
<td>Super1603</td>
<td>100</td>
<td>3.5</td>
<td>1200</td>
<td>4200</td>
<td>25.12</td>
<td>0.0060</td>
<td>15.85</td>
<td>0.21</td>
<td>Voegelo</td>
</tr>
<tr>
<td><strong>Micro-surfacing Machineries</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>M206</td>
<td>*74:186</td>
<td>2400</td>
<td>3600</td>
<td>41.70</td>
<td>0.0116</td>
<td>30.70</td>
<td>0.417</td>
<td>Bergkamp</td>
<td></td>
</tr>
<tr>
<td>M210</td>
<td>*74:224</td>
<td>2400</td>
<td>3600</td>
<td>42.40</td>
<td>0.0118</td>
<td>31.75</td>
<td>0.424</td>
<td>Bergkamp</td>
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<tr>
<td><strong>Milling machine</strong></td>
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<td></td>
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<td></td>
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<tr>
<td>PL2100S</td>
<td>447</td>
<td>2.1</td>
<td>1800</td>
<td>3780</td>
<td>104.77</td>
<td>0.0277</td>
<td>73.45</td>
<td>0.97</td>
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<tr>
<td>W120F</td>
<td>227</td>
<td>1</td>
<td>1800</td>
<td>1800</td>
<td>53.20</td>
<td>0.0296</td>
<td>78.33</td>
<td>1.04</td>
<td>Dynapac</td>
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<tr>
<td><strong>Double Drum Steel Rollers (6 and 3 passes)</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CC421</td>
<td>80</td>
<td>*1.42</td>
<td>4000</td>
<td>946</td>
<td>20</td>
<td>0.0190</td>
<td>50.35</td>
<td>0.684</td>
<td>Dynapac</td>
</tr>
<tr>
<td>CC421</td>
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<td>*1.42</td>
<td>4000</td>
<td>1893.3</td>
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<td>0.0106</td>
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<td>0.371</td>
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<td><strong>Pneumatic Tire Rollers (6 passes)</strong></td>
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<tr>
<td>CP142</td>
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<td>*1.50</td>
<td>10000</td>
<td>2493.3</td>
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<td>15.41</td>
<td>0.204</td>
<td>Dynapac</td>
</tr>
<tr>
<td>CP274</td>
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<td>*1.96</td>
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<td>0.0050</td>
<td>13.18</td>
<td>0.175</td>
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<tr>
<td><strong>Vibrating Smooth Wheeled Rollers (6 passes)</strong></td>
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<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>213DH-S</td>
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<td>*1.81</td>
<td>4000</td>
<td>1207.0</td>
<td>18.7</td>
<td>0.0155</td>
<td>41.06</td>
<td>0.544</td>
<td>Bomag</td>
</tr>
<tr>
<td>177DH-S</td>
<td>76</td>
<td>*1.43</td>
<td>4000</td>
<td>952.0</td>
<td>28.2</td>
<td>0.0297</td>
<td>78.61</td>
<td>1.041</td>
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<td><strong>Motor Grader</strong></td>
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<td></td>
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<td></td>
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<td>120K</td>
<td>111</td>
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<td>2007</td>
<td>27.30</td>
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<td>36.05</td>
<td>0.48</td>
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<tr>
<td>160H</td>
<td>134</td>
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<td>2247</td>
<td>38.50</td>
<td>0.0171</td>
<td>45.40</td>
<td>0.60</td>
<td>CAT</td>
<td></td>
</tr>
</tbody>
</table>

*a The first value is mixer engine rating and the second value is truck engine rating
b The effective width shown for rollers is the 85% of the actual roller width
3.2.3. Embodied Energy and Carbon Emissions for Preservation Treatments

The last step in our LCA is to calculate the embodied energy and emissions for the selected preservation treatments. Considering the materials, activities and equipment/machinery involved, it was easy to calculate the emissions and energy consumption. The fractions or the mix designs of each treatment were established in order to determine the quantity (kg) of each component per ton. Based on this value together with the density and thickness, it was possible to quantify each element in tons per square meter. The procedure for Chip seal and Micro-surfacing was slightly different as the aggregate application rate from the mix design is in kg/m², so converting to tons per square meter was easy. The values obtained were then multiplied by the emissions and energy consumption data listed in the previous section to give the total emissions and energy consumption for each preservation treatment. It is important to note that the above explained procedure is not necessary in regard to equipment/machinery calculations as their data is already expressed per square meter [9, 11, 21, 50].

The mix designs for HMA asphalt, reconstruction and chip seal with their respective thicknesses is based on data from projects in Zambia while for micro-surfacing, data from the Georgia Department of Transportation was used [19]. A spreadsheet tool was created for easy calculation and automation in taking into account different possible treatment combinations. The specifics of each treatment case are shown and discussed below.

a) Thin Hot Mix Asphalt Overlay

A typical mix design was chosen clearly showing the percentages of bitumen, aggregates and filler in the asphalt. The intervention thickness is 40 mm asphalt concrete; this will greatly help us calculate the total volume of materials used per square meter of the treatment. The mix design is 4.8% of 60/70 Penetration Grade Bitumen, 1% Hydrated Lime, 50.2% Pit Run Aggregate and 44% Crushed Aggregate. The production of HMA follows the usual procedure consisting of drying the aggregates (coarse and fine) by heating at high temperatures then mixing the materials (bitumen, aggregate (coarse and fine) and filler). Results show carbon emissions of 5.28 kg/m² and embodied energy of 74.83 MJ/m² as Table 3 highlights the calculation procedure for this type of treatment.

b) Micro-surfacing

A similar procedure similar to the one outlined above was used for applying a micro-surfacing mixture on a square meter of a road pavement. The design is a Type III Aggregate gradation containing 81.6%, 7.4% of Modified Emulsion (3.5% SBR), 1% Cement Filler and 10% of Potable Free Water. The objective is to get a 19mm thick surface layer which is capable of providing adequate surface protection and maintaining a high pavement serviceability level. Results show carbon emissions of 2.48 kg/m² and embodied energy of 39.32 MJ/m².

<table>
<thead>
<tr>
<th>Table 3. Embodied energy and carbon emissions of preservation treatments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>HMA Mix Design Input Parameters</strong></td>
</tr>
<tr>
<td>Bulk Specific Density (ton/m³)</td>
</tr>
<tr>
<td>2.48</td>
</tr>
<tr>
<td>2.80</td>
</tr>
<tr>
<td>4.8</td>
</tr>
<tr>
<td>0.04</td>
</tr>
<tr>
<td>0.0992</td>
</tr>
<tr>
<td>% of Individual in Mix</td>
</tr>
<tr>
<td>100</td>
</tr>
<tr>
<td>0.01</td>
</tr>
<tr>
<td>0.048</td>
</tr>
<tr>
<td>Pavement Thickness (m)</td>
</tr>
<tr>
<td>0.04</td>
</tr>
<tr>
<td>(ton/m²)</td>
</tr>
<tr>
<td>0.0992</td>
</tr>
<tr>
<td>Thin HMA Overlay 0.04 m</td>
</tr>
<tr>
<td>Quantity [ton/m²]</td>
</tr>
<tr>
<td>0.0048</td>
</tr>
<tr>
<td>Carbon Emission [kg/ton]</td>
</tr>
<tr>
<td>303.6</td>
</tr>
<tr>
<td>4832.8</td>
</tr>
<tr>
<td>4.8</td>
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<tr>
<td>0.0436</td>
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<tr>
<td>Embodied Energy [MJ/ton]</td>
</tr>
<tr>
<td>6.3</td>
</tr>
<tr>
<td>38.62</td>
</tr>
<tr>
<td>28</td>
</tr>
<tr>
<td>0.0498</td>
</tr>
<tr>
<td>0.0436</td>
</tr>
<tr>
<td>0.0436</td>
</tr>
<tr>
<td>Total Carbon Emission [kg/m²]</td>
</tr>
<tr>
<td>1.45</td>
</tr>
<tr>
<td>0.27</td>
</tr>
<tr>
<td>0.24</td>
</tr>
<tr>
<td>0.001</td>
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<tr>
<td>Total Embodied Energy [MJ/m²]</td>
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<td>23.01</td>
</tr>
<tr>
<td>1.69</td>
</tr>
<tr>
<td>1.39</td>
</tr>
<tr>
<td>Fuel Usage [l/h]</td>
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</tr>
<tr>
<td>0.01543</td>
</tr>
<tr>
<td>0.05035</td>
</tr>
<tr>
<td>0.01541</td>
</tr>
<tr>
<td>0.491</td>
</tr>
<tr>
<td>0.2</td>
</tr>
<tr>
<td>0.684</td>
</tr>
<tr>
<td>0.204</td>
</tr>
</tbody>
</table>

Materials

<table>
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<tr>
<th>Bitumen</th>
<th>0.0048</th>
<th>303.6</th>
<th>4832.8</th>
<th>1.45</th>
<th>23.01</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crushed Aggregate</td>
<td>0.0436</td>
<td>6.3</td>
<td>38.62</td>
<td>0.27</td>
<td>1.69</td>
</tr>
<tr>
<td>Pit-run Aggregate</td>
<td>0.0498</td>
<td>4.8</td>
<td>28</td>
<td>0.24</td>
<td>1.39</td>
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<tr>
<td>Hydrated Lime Filler</td>
<td>0.001</td>
<td>245.0</td>
<td>1244</td>
<td>0.24</td>
<td>1.23</td>
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<tr>
<td>HMA Production</td>
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<td>19.0</td>
<td>299.5</td>
<td>1.88</td>
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<tr>
<td>Tackcoat (SS60)</td>
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<td>225.0</td>
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<td>0.23</td>
<td>3.19</td>
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</table>

Machinery/Equipment

<table>
<thead>
<tr>
<th>Tackcoat Sprayer (HM 10HD)</th>
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<th>0.491</th>
</tr>
</thead>
<tbody>
<tr>
<td>Paver (Dynapac DF145C)</td>
<td>0.01543</td>
<td>0.2</td>
</tr>
<tr>
<td>Roller Dynapac CC421</td>
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<td>0.684</td>
</tr>
<tr>
<td>Roller (Pneumatic)</td>
<td>0.01541</td>
<td>0.204</td>
</tr>
</tbody>
</table>
c) Chip Seal with a Fog Seal

A double surface dressing with 13.2 mm and 6.7 mm crushed aggregates was chosen with an aggregate application rate of 1.632 kg/m$^2$ and 15 kg/m$^2$ respectively and a 80/100 penetration grade bitumen application rate of 0.8 l/m$^2$ was used as a binder. A fog seal which is simply an emulsion is applied at a rate of 0.5 l/m$^2$ and is eventually sprayed on top of the chip seal to complete the treatment. Results show carbon emissions of 1.68 kg/m$^2$ and embodied energy of 23.07 MJ/m$^2$.

d) Mill and Fill

This treatment option involves the milling of 20 mm of the asphalt layer and then resurfacing it with 50 mm new asphalt. The procedure and calculations are the same with the Thin HMA overlay. Results show carbon emissions of 6.73/kg/m$^2$ and embodied energy of 95.21 MJ/m$^2$.

e) Major Reconstruction

This is a last resort intervention when during the pavement design life no maintenance strategy was employed on the pavement. It consists of milling the 40/50 mm existing AC layer then milling/scarifying the 150/200 mm underlying base layer and replacing them with a new 150 mm gravel sub-base and a 120mm cemented base or a 150/200 mm crushed stone as the new base and finally laying a new 50 mm AC layer. This is done in order to achieve a pavement structural number consistent with the new traffic loadings at the time of rehabilitation. Results show carbon emissions of 13.54/kg/m$^2$ and embodied energy of 160.81 MJ/m$^2$.

Figure 3 below summarizes and shows the embodied energy and carbon emissions of a 1km 3.5m lane road.

<table>
<thead>
<tr>
<th>Treatment</th>
<th>Laying (kg/m$^2$)</th>
<th>Materials (kg/m$^2$)</th>
<th>Transport (kg/m$^2$)</th>
<th>Total (kg/m$^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transport HMA (10 km)</td>
<td>0.0617</td>
<td>0.901</td>
<td>0.061</td>
<td>0.894</td>
</tr>
<tr>
<td>Transport Bitumen (1500 km)</td>
<td>0.0617</td>
<td>0.901</td>
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<td>6.435</td>
</tr>
<tr>
<td>Transport Tack Coat (1500 km)</td>
<td>0.0617</td>
<td>0.901</td>
<td>0.093</td>
<td>1.352</td>
</tr>
<tr>
<td>Transport C. Aggregates (50 km)</td>
<td>0.0617</td>
<td>0.901</td>
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<td>1.966</td>
</tr>
<tr>
<td>Transport P. Aggregates (50 km)</td>
<td>0.0617</td>
<td>0.901</td>
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<td>2.243</td>
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<tr>
<td>Transport H. Lime (150 km)/km</td>
<td>0.0617</td>
<td>0.901</td>
<td>0.009</td>
<td>0.134</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>5.28</strong></td>
<td><strong>74.83</strong></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

![Figure 3](image-url)
Life Cycle Cost Calculation

The LCCA process either probabilistic or deterministic is initiated after a road section has been earmarked to be worked on and a range of possible alternatives have been identified in the quest to improve the pavement. Then the activity timing follows; this timing of treatments should be based on existing performance records or literature to achieve maximum cost benefits. The effectiveness of the treatment is also a fundamental input for a good LCCA [5, 17-26]. After determining the activity timing and effectiveness of selected treatments as listed in Table 4, the estimation of costs follows. In this paper only the costs demonstrating differences between alternatives are considered. These are the cost of the individual treatments and are obtained from the World Bank ROCKS software since the World Bank is one of the biggest road projects funders and moreover this data shows less variability and is similar to costs quoted in Zambia [52]. The cost has been converted to its April, 2019 value using the inflation index suggested by the United States Bureau of Statistics and then converted to Zambian Kwacha (ZMW) [53]. Zambia as a developing country and according to the World Bank uses 12% as a discount rate. Finally, the computation of the life cycle cost using the formulas given in the previous section is done and summarized in Table 5.

### Table 4. Effective treatment life and treatment cost

<table>
<thead>
<tr>
<th>Treatment Type</th>
<th>Effective Treatment Life (Years)</th>
<th>Treatment Cost (US$/m²)</th>
<th>Treatment Cost (ZMW/m²)</th>
<th>Cost (ZMW/km) with 3.5 m of width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full Depth Recon.</td>
<td>20</td>
<td>46.24</td>
<td>568.29</td>
<td>1989013.60</td>
</tr>
<tr>
<td>Mill &amp; Fill Overlay (MF)</td>
<td>10</td>
<td>19.52</td>
<td>239.90</td>
<td>839652.80</td>
</tr>
<tr>
<td>HMA Overlay (TO)</td>
<td>7</td>
<td>12.63</td>
<td>155.22</td>
<td>543279.45</td>
</tr>
<tr>
<td>Micro-surfacing (MS)</td>
<td>6</td>
<td>4.96</td>
<td>60.96</td>
<td>213354.40</td>
</tr>
<tr>
<td>Chip &amp; Fog Spray (CF)</td>
<td>5</td>
<td>4.20</td>
<td>51.62</td>
<td>180663.00</td>
</tr>
</tbody>
</table>

### Table 5. Cost, NPV and EUAC of different treatment scenarios

<table>
<thead>
<tr>
<th>Treatment Type</th>
<th>Year of Application</th>
<th>Cost (ZMW/km) with 3.5m lane</th>
<th>NPV (ZMW/km) with 3.5m lane</th>
<th>EUAC (ZMW/km) with 3.5m lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>HMA Thin Overlay (TO)</td>
<td>9 and 17</td>
<td>1086558.90</td>
<td>2264048.82</td>
<td>277573.83</td>
</tr>
<tr>
<td>Mill &amp; Fill Overlay (MF)</td>
<td>12</td>
<td>839652.80</td>
<td>2204535.68</td>
<td>271775.88</td>
</tr>
<tr>
<td>Micro-surfacing (MS)</td>
<td>7 and 13</td>
<td>426708.80</td>
<td>2134418.89</td>
<td>263131.86</td>
</tr>
</tbody>
</table>
4. Results and Discussion

4.1. Energy and Carbon Emission

The Figure 4 below shows that full depth reconstruction, a combination of thin overlay and mill and fill and thin overlay consume the largest amount of embodied energy of 788GJ, 595.1GJ and 523.8GJ respectively. These values are a bit high due to a simple reason that the production process and transportation of materials is the highest in the energy chain. The treatments that require a lot of aggregate heating and having the largest quantity of materials per unit area use the largest amounts of energy. The results show embodied energy amounts of 3 to 4 times that of chip + fog seal, micro-surfacing and a combination of chip + fog seal and micro-surfacing treatment having values of 161.5GJ, 218.4GJ and 275.2GJ respectively. The embodied energy consumption difference between the greatest and the least treatment option is 646.3GJ which can be used on 4km of road pavement using the least treatment preservation option.

Taking into account the carbon emissions, the figure shows that full depth reconstruction, a combination of thin overlay with mill and fill and thin overlay are the largest emitters with values of 66.3tons, 42tons and 37tons respectively. The values are in excess of 3-6 times the three least treatment emitters of chip + fog seal, micro-surfacing and a combination of chip + fog seal and micro-surfacing having values of 11.8tons, 14.6tons and 17.3tons respectively. The carbon emission difference between the most and the least treatment option is 54.5tons which can be used on 5km of road pavement using the least treatment preservation option. Generally, chip + fog seals showed the lowest variability with micro-surfacing following it. The carbon emissions and energy for micro-surfacing can be further reduced if the percentage of plastics in the modified binders is reduced. The modified emulsions have high percentage of carbon emissions and have thus increased the overall performance of micro-surfacing in terms of emissions impact to the environment. Research and development in better understanding of the binder will provide a basis for better preservation design optimization, hence greatly reducing the energy consumption and carbon emissions.

<table>
<thead>
<tr>
<th>Treatment Options</th>
<th>Embodied Energy (GJ)</th>
<th>Carbon Emission (Tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Full Depth Reconstruction</td>
<td>788.0</td>
<td>66.3</td>
</tr>
<tr>
<td>MS &amp; TO</td>
<td>756633.85</td>
<td>11.8</td>
</tr>
<tr>
<td>MS &amp; TO</td>
<td>723942.45</td>
<td>14.6</td>
</tr>
<tr>
<td>CF &amp; TO</td>
<td>739424.45</td>
<td>17.3</td>
</tr>
<tr>
<td>MS &amp; CF</td>
<td>394017.40</td>
<td>11.8</td>
</tr>
<tr>
<td>MS &amp; MF</td>
<td>1053007.20</td>
<td>14.6</td>
</tr>
<tr>
<td>CF &amp; MF</td>
<td>1020315.80</td>
<td>17.3</td>
</tr>
</tbody>
</table>
4.2. Life Cycle Cost

The results obtained from the LCCA after the necessary calculations are shown in Figure 5. The NPV shows marked differences among the costs of the scenarios.

Based on the costs and NPV, the figure shows that the most economical were the chip + fog seal and micro-surfacing then followed by a treatment whose options included the above two treatments and the least economical was a combination of mill and fill with thin overlay and then full depth reconstruction. Chip + Fog seal has the lowest life cycle NPV of ZMW2126915.47 then followed by micro-surfacing with ZMW2134418.89. The most expensive treatment
was full depth reconstruction at ZMW3292821.96 while a combination of mill and fill with thin overlay followed at ZMW2361837.87. It is worth noting that the three most economical treatments show an LCC of 30-40% lower than the full depth reconstruction. Agencies should bear in mind that when the LCC among treatment options show a difference of less than 10%, it would be wise to consider the treatment options to be equivalent or consider the work zone traffic influence and other parameters as tie breakers.

5. Conclusion
An economic and environmental analysis among selected pavement preservation treatments through a case study was calculated and compared. Based on the costs and NPV, results showed that the most economical were the chip + fog seal and micro-surfacing and the least economical was full depth reconstruction and a combination of mill and fill and thin overlay. Although the LCCA appears to show that chip + fog seal is overall the best preservation treatment, the selection of the treatments depends on different factors based on the characteristics of each treatment option. These factors include environmental conditions, availability of required materials, as well as traffic volumes or loads. The environmental analysis showed that different treatments require different amounts of embodied energy and emit carbon differently during the analysis period of 30 years. Full depth reconstruction and a combination of thin overlay and mill and fill treatment options have the highest embodied energy consumption while chip + fog seal and a combination of micro-surfacing and chip + fog seal utilize the least amounts of embodied energy. Similar result patterns apply to carbon emissions with full depth reconstruction being the biggest emitter and chip + fog seal as the least emitter. The results show and agree with other literature that treatments that do not require aggregate heating and those having the lowest amount of materials per unit area use the least amount of energy and have the lowest carbon emissions [9-11].

The raw material extraction process is the highest in the energy and carbon emission chain. This highlights the need to have the quarry site, treatment plants and construction site to be in close proximity in order to avoid excess energy use and carbon emissions which result in high economic costs. In light of the above results, an expertly defined LCA system for preservation treatments of roads is the only way forward to the path of a “green” procurement and operational use and recycling of materials in the construction of roads. This system can also be made useful in the implementation of the carbon tax in the road construction sector.

As this study only considered selected preventive maintenance treatments and the life cycle cost and assessment scope was narrowed, it is recommended that elements of user costs including accident costs, work zone costs and environmental costs be extended for a better estimation of the total life cycle costing. Additionally, vehicle emissions derived from pavement condition and work zones should also be considered in accounting the carbon emissions from the application of preservation treatments. Expensive and deeper strategies of rehabilitation can be avoided by the application of perpetual pavements which would require less maintenance interventions thus reducing the embodied energy, carbon emissions and economical cost of treatments. Finally, it should be recommended to track the maintenance strategies carried out as a result of the application of this model so as to validate, adjust or dismiss the parameters considered.

6. Conflicts of Interest
The authors declare no conflict of interest.

7. References


Behaviour of Soft Clayey Soil Improved by Fly Ash and Geogrid under Cyclic Loading

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Abstract

The effect of Cyclic loading on the foundation behaviour of many engineering structures presents more important and related to many problems in geotechnical engineering. Especially when construction on soft ground area which represent one of the major concerns in geotechnical engineering. This paper is conducted to investigate the influence of using several improving techniques as (fly ash, Geo-grid, fly ash and Geo-grid) on the behavior of soft clayey soil subjected to cyclic loading. A total of twenty four models have been tested which consists of a wide domain of boundary conditions, such as untreated model, Geo-grid reinforced models, fly ash treated models and models treated with fly ash incorporated with Geo-grid were conducted by varying parameters such as, footing elevations, test velocity and number of geogrid layers. The analysis demonstrates that the settlement behaviour of footing resting on treated models with fly ash and two Geo-grid layers perform better than other improving techniques. Also observed there was an increase in settlement, which corresponds to the increase in test velocity from 6 to 9 mm/sec. Furthermore, it was conducted that the more depth of footing the soil settlement decreases. In general, when other factors remaining constant, the bearing capacity of soil goes on increasing when the depth increased.

Keywords: Soft Soil; Fly Ash; Geogrid; Soil Treatment; Stabilization; Cyclic Loading.

1. Introduction

Many regions in Iraq contain very soft clays especially in south, which have undesirable geotechnical properties such as, low bearing capacity, high compressibility. And because of the rapid economic development and construction of infrastructure particularly transportation infrastructure (e.g., high speed railways, airports, expressways, subways, and ports) in which the load characteristics in most cases as cyclic loading. However, there is an increasing demand for studies of the cyclic behavior of clay and methods of improvement. In the absence of a suitable ground improvement, the soft clay deposits clays can sustain excessive settlement, high excess pore-water pressures during cyclic loading. This have negatively effects on the stability of buildings [1]. Reinforcing soft ground by installing geogrid and fly ash well established technique practiced worldwide as well as suitable methods to enhance the geotechnical properties of soil. Settlement of structures built on improved soils with fly ash decreases and the time required for reaching the final settlement is reduced. Fly ash can be used in soil to get improvement in shear strength, cohesion and improvement in the bearing capacity [2]. Also, the inclusion of geogrid leads to increase bearing capacity of shallow foundation by placing one or several layers. Furthermore, it becomes more stable and the factor of safety against bearing capacity
failure is improved. The reinforced sand beds perform better than the unreinforced sand beds regardless of the number of reinforcement layers and spacing of the reinforcement under repeated loads [3]. Cyclic loading and its influences towards foundation performance are of the highest importance and associated with many problems in geotechnical engineering. The foundation of structures require the special attention of the civil engineer when subjected cyclic loads in addition to static loads. The response of earth structures to dynamic stress applications, such as those produced by machine loads, wave loads and other low frequency loads are finding increased application in civil engineering practice. There are several sources of cyclic loading, such as traffic (high-speed train), industrial sources (crane rails, machine foundation), wind and waves (on – shore and off – shore wind power plants, coastal structures) or repeated filling and emptying processes (Watergates tanks and silos). In addition, cyclic loading imposes to soil during earthquake events that slipped the adjacent tectonic plates lead to a propagation of shear waves that shear waves induce a cyclic shearing of the soil [4].

The use of fly ash and geogrid reinforcements for improving load-bearing capacity of foundation has been widely studied by many researchers, such as Honnalli and Rakaraddi (2015) studies, which presents the efficacy of multilayer reinforcements in improving the load-bearing capacity when incorporated within the body of fly ash embankment. An increase in load bearing capacity due to the incorporation of reinforcement layers in the model slope were observed in the laboratory tests [5]. Hotti et al. (2014) Conducted laboratory model tests on square footings supported by geogrid reinforced sand bed and subjected to incremental loading and unloading conditions. The results shows that the value of ultimate bearing capacity for reinforced sand are greater than unreinforced sand bed [6]. Also, Saisubramanian et al. (2019) studied the effect of coir geotextile reinforcement on the vertical stress distribution in the sand. The test results showed the inclusion of reinforcement can redistribute the applied footing load to a more uniform pattern, hence reducing the stress concentration which will result reduced settlement [7]. Sridhar and Kumar (2018) reported that with use two layers of Geo-textiles on soft clay led to that the shear strength was enhanced substantially. This study investigates the influence of using improving techniques. on the behavior of soft clayey soil subjected to cyclic loading by achieving a series of model footing tests consists of a wide domain of boundary conditions (untreated and treated with fly ash in addition to the geogrid layers) and varying parameters, such as footing elevations, test velocity and number of geogrid layers [8].

2. Materials and Methods

2.1. Soil

A brown clayey soil was collected from Al-Nahrawan city east of Baghdad. Standard examinations were conducted to decide the properties of the soil used. Details are listed in Table 1. Grain size distribution of the soil used which represent the following percentages: 7.02 % sand, 43.98% silt and 49% clay as illustrated in Figure 1. According to the USCS, the soil is classified as CL.

![Figure 1. Grain size distribution of the soil used](image-url)

Table 1. Physical properties of the soil used

<table>
<thead>
<tr>
<th>Property index</th>
<th>Value index</th>
<th>Standard index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit (LL) %</td>
<td>32</td>
<td>ASTM D4318</td>
</tr>
<tr>
<td>Plastic limit (PL) %</td>
<td>17</td>
<td>ASTM D4318</td>
</tr>
<tr>
<td>Plasticity index (PI) %</td>
<td>15</td>
<td>ASTM D4318</td>
</tr>
<tr>
<td>Specific gravity (Gs)</td>
<td>2.69</td>
<td>ASTM D854</td>
</tr>
<tr>
<td>Gravel (larger than 2mm) %</td>
<td>0</td>
<td>ASTM D422</td>
</tr>
<tr>
<td>Sand (0.06 to 2mm) %</td>
<td>7.02</td>
<td>ASTM D422</td>
</tr>
</tbody>
</table>
Silt (0.005 to 0.06mm) % 43.98 ASTM D422
Clay (less than 0.005mm) % 49 ASTM D422
Maximum dry unit weight (kN/m$^3$) 16.7 ASTM D1557
Optimum moisture content (%) 21 ASTM D1557

USCS CL

2.2. Geogrid

The geogrid used in all tests was manufactured by Al-Latifia Factory for plastic mesh having the engineering properties shown in Table 2 as provided by the manufacturing company [9, 10]. The sheets of geogrid were used from test to test but they were replaced whenever any of the strands become visibly overstressed. Figure 2 illustrate the Geo-grid used.

Table 2. Engineering properties of the geogrid used [11] (A) Physical, chemical and biological properties (B) Dimensions and technical properties

<table>
<thead>
<tr>
<th>Property</th>
<th>Test method</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structure</td>
<td>Extruded geogrid</td>
<td></td>
</tr>
<tr>
<td>Mesh type</td>
<td>Square</td>
<td></td>
</tr>
<tr>
<td>Standard colour</td>
<td>Green</td>
<td></td>
</tr>
<tr>
<td>Polymer type</td>
<td>HDPE</td>
<td></td>
</tr>
<tr>
<td>Packing</td>
<td>Rolls</td>
<td></td>
</tr>
<tr>
<td>Chemical resistance</td>
<td>ASTM D1603</td>
<td></td>
</tr>
<tr>
<td>Biological resistance</td>
<td>The product is not affected by micro orogenesis.</td>
<td></td>
</tr>
<tr>
<td>Sunlight resistance</td>
<td>The addition of suitable stabilizers limits the attack from UV light. The material can be expected to have a life of over 5 years when exposed, without a loose of more than 20% of the product strength in a temperature climate.</td>
<td></td>
</tr>
<tr>
<td>Temperature stability</td>
<td>The material is stable within a temperature range of -60 °C to 100 °C, but with a reduced strength at elevated temperature.</td>
<td></td>
</tr>
<tr>
<td>UV stabilizer</td>
<td>Added with color</td>
<td></td>
</tr>
</tbody>
</table>

(B)

<table>
<thead>
<tr>
<th>Property</th>
<th>Test method</th>
<th>Unit</th>
<th>Data</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aperture size</td>
<td>ISO 9864</td>
<td>mm*mm</td>
<td>6*6</td>
</tr>
<tr>
<td>Mass per unit area</td>
<td>ISO 9864</td>
<td>g/m$^2$</td>
<td>363</td>
</tr>
<tr>
<td>Roll width</td>
<td>ISO 9864</td>
<td>m</td>
<td>1</td>
</tr>
<tr>
<td>Roll length</td>
<td></td>
<td>m</td>
<td>30</td>
</tr>
<tr>
<td>Tensile strength at 2 %</td>
<td>ISO 9864</td>
<td>KN/m$^2$</td>
<td>4.3</td>
</tr>
<tr>
<td>Tensile strength at 5 %</td>
<td>ISO 9864</td>
<td>KN/m$^2$</td>
<td>7.7</td>
</tr>
<tr>
<td>Peak tensile strength</td>
<td></td>
<td>KN/m$^2$</td>
<td>13.5</td>
</tr>
<tr>
<td>Yield point elongation</td>
<td>ISO 9864</td>
<td>%</td>
<td>20.0</td>
</tr>
<tr>
<td>Aperture size</td>
<td>ISO 9864</td>
<td>mm*mm</td>
<td>6*6</td>
</tr>
<tr>
<td>Mass per unit area</td>
<td></td>
<td>g/m$^2$</td>
<td>363</td>
</tr>
</tbody>
</table>

Figure 2. The Geo-grid used
2.2. Fly Ash

The fly ash used is obtained from the south of Baghdad thermal power plant. The chemical compositions are given in Table 3. According to ASTM C 618 [12], this fly ash can be classified as a Class C fly ash.

<table>
<thead>
<tr>
<th>Composition</th>
<th>Value (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO$_2$</td>
<td>32.16</td>
</tr>
<tr>
<td>Fe$_2$O$_3$</td>
<td>6.13</td>
</tr>
<tr>
<td>Al$_2$O$_3$</td>
<td>18.37</td>
</tr>
<tr>
<td>TiO$_2$</td>
<td>&lt;0.01</td>
</tr>
<tr>
<td>CaO</td>
<td>28.23</td>
</tr>
<tr>
<td>MgO</td>
<td>8.16</td>
</tr>
<tr>
<td>CO$_3$</td>
<td>0.34</td>
</tr>
<tr>
<td>Na$_2$O</td>
<td>2.83</td>
</tr>
<tr>
<td>K$_2$O</td>
<td>3.19</td>
</tr>
</tbody>
</table>

3. Loading Machine

The loading machine consists of several parts [13]. As illustrated in Figure 2 which it (Loading jack, Steel Frame, Footing models, steel container).

4. Model Footing and Container

Based on the recommendations given by Mohammed (2018) [2], steel ring footing was used in this study with inner diameter of 40 mm and outer diameter of 100 mm and thickness of 20 mm. Steel container is used with inner dimensions of 600×600 mm and depth of 500 mm. The container is made as one piece with a thickness of 6 mm steel plate.

The settlement of the footing during the application of cyclic load was measured by using Laser LVDT. It can measure absolute distances up to 2m. Table 3 listed the specification of the Laser LVDT used. In addition, the system of data acquisition was utilized so that all data could be scanned and recorded automatically by using computer and data logger.
Table 3. Specification of the laser LVDT used

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Supply voltage</td>
<td>2.6 to 3.5 V</td>
</tr>
<tr>
<td>Global current</td>
<td>15 mA</td>
</tr>
<tr>
<td>Consumption</td>
<td></td>
</tr>
<tr>
<td>Ultrasonic frequency</td>
<td>Up to 400 kHz</td>
</tr>
<tr>
<td>Maximal range</td>
<td>200 cm</td>
</tr>
<tr>
<td>Minimal range</td>
<td>3 cm</td>
</tr>
<tr>
<td>Resolution</td>
<td>1 cm</td>
</tr>
<tr>
<td>Trigger pulse width</td>
<td>10 μs</td>
</tr>
<tr>
<td>Outline dimension</td>
<td>4.4 x2.4 x 1.0 mm</td>
</tr>
</tbody>
</table>

5. Model Preparation

The clayey soil mixed with predetermined amount of water which corresponding to $c_u = 7$ kPa. To get uniform moisture content the wet soil was kept inside tightened polythen bags for a period of four days before use. The mixing operation was conducted using a large mixer (120 L) manufactured for this purpose. After that, the moist soil was placed in five layers inside the container. And pressed gently with a wooden tamper in order to remove entrapped air. Figure 3 illustrates model preparation.

The fly ash content was 20%, which was optimum percentage depending on previous research, which defined by the ratio of the weight of the fly ash to the dry weight of the natural clayey soil (soft Soil) expressed as a percentage. To achieve that by mixing 25 Kg of dry soil with 5 Kg of fly ash and desired water content. The geogrid layers were placed at desired depth which equal to 5 cm and 7.5 cm (0.5 D, 0.75 D where D = external diameter of footing) Measured from the base of footing.

![Figure 3. Model preparations](image)

6. Results and Discussion

- The number of model tests is 24, which are arranged in four groups as following:
- Four untreated models with ring footing ($D_{in}/D_{out}=0.4$) ($D_{in}$ is the internal diameter and $D_{out}$ is the external diameter of footing used) placed on soft saturated bed of clay with $c_u=7$ kPa.
- Four models were prepared and testing with fly ash only with two footing elevations which ($D_f=0$ and $D_f=100$mm) and two velocities that ($V_1=9$ mm/sec and $V_2=6$ mm/sec).
- Eight models were conducted with reinforcement geogrid of variable geogrid depths (0.5 D and 0.75 D; where, D = the diameter of footing) and with two footing elevations ($D_f=0$ and 100 mm) and two velocities ($V_1=9$ mm/sec and $V_2=6$ mm/sec).
- Eight models were tested with reinforcement geogrid (with the same depth and velocities mentioned above) and fly ash. For all test models the failure was defined as the load causing a settlement corresponding to 10% of the diameter of footing based on Terzaghi (1943) [14]. In this study, the settlement with time was investigated.
6.1. Model Test on Untreated Soil

Figures 4 and 5 demonstrate the variation of settlement with time for untreated models with variable footing depths (footing at surface and footing at depth) for velocity test equal to 9 and 6 mm/sec respectively. From these figures, it can be seen that the more the depth of footing the soil settlement decrease as presented in figures. In general, when other factors are remaining constant, the bearing capacity of the soil goes on increasing when the depth of the foundation increases. The total settlement of a footing continues to increase during the time reaching a maximum value at the end of test. Moreover, there is an increase in settlement with increasing test velocity which attributed to in case of increase test velocity the soil particles do not have enough time to arrange themselves in denser state. This is in agreement with Fattah et al. (2017) results [4].

![Figure 4. Settlement versus time for untreated models with different footing depth and velocity=9 mm/sec](image)

![Figure 5. Settlement versus time for untreated models with different footing depth and velocity=6 mm/sec](image)
6.2. Model Test on Treated Soil with Fly Ash

Figures 6 and 7 represent the variation of settlement with time for treated with fly ash and untreated models. From these Figures, it can be noticed with the addition of fly ash there is substantial reduction in settlement of foundation. That is attributed to the pozzolanic reaction between the calcium aluminates within the fly ash and the minerals of soil, which dissolving into water to form calcium-silicate and calcium-aluminate gels, which turn the soil into hardened solid with high strength and stiffness. The improvement in settlement measured at the end of test where more than 50 % when footing placed at surface (D_f = 0 mm), and more than 40% for footing placed at depth (D_f =100 mm) the results are in agreement with Khan et al. (2008) results [15]. Who adopted that “At lower intensity of load and higher number of cycles, The strength of layered system of soil-fly ash matrix increases due to interlocking of fly ash particles and binding effect of interfaces of soil and fly ash layers”, Also it can be observed an increase in settlement measured at the end of test from 7.4 mm to 8 mm when footing placed at surface and from 5.8 to 6.2 mm when footing placed at depth. This corresponds to the increase in test velocity from 6 mm/sec to 9 mm/sec.

![Figure 6. Settlement versus time for untreated and treated with fly ash models with different footing depth and velocity=9 mm/sec](image6)

![Figure 7. Settlement versus time for untreated and treated with fly ash models with different depth and velocity=6 mm/sec](image7)
6.3. Model Test on Treated Soil with Geogrid

Figures 8 to 11 present the relationship between time and settlement for untreated and treated with 1 and 2 geogrid layer models. Obviously as seen in these figures, the amount of settlement increased with time progress, the settlement was slightly lower for the case of reinforced clay with one geogrid layer than unreinforced clay. In contrast, it can be observed that in case of using double geogrid layers, there is an improving in the settlement measured at the end of the test, which was about 16.58 – 10 mm and 12.89 – 8.1 mm. For models footings placed at the surface (D_f = 0) and at the depth (D_f =100 mm) respectively with test velocity = 9 mm/sec. This reduction is likely to happen because of, interlocking between the clay and geogrid, when the soil interlocks in the geogrid and stress is applied. The stress is transmitted to the rib of geogrid more precisely the stress transmitted to the longitudinal ribs through the junction leading to increase bearing capacity subsequently and enhance settlement performance. This is consistent with Hotti et al. (2014) findings who noted that there was further increases in the load carrying capacity with the inclusion of geogrid reinforcement [6]. In addition, the use of the geogrid reinforcement leads to better performance from the point of view of Cu improvement as well as settlement reduction. This also observed by Das and Shin (1994) [16], who studied the permanent settlement of a surface strip foundation supported by geogrid-reinforced saturated clay and subjected to a low-frequency cyclic load. The results present that “Full depth geogrid reinforcement may reduce the permanent settlement of a foundation by about 20% to 30% compared to one without reinforcement”. Sudhakar and Sandeep (2016) studied the effect of geocell reinforcement on bearing capacity of ring footing and circular footing under vertical loading and cyclic loading using experimental approach. The results showed that “footing performance due to cyclic loading is better for geocell reinforced soil than that of unreinforced soil” [17]. Also for the same condition, the improvement in ultimate bearing capacity increases with increase of reinforcement layers that conducted by Zidan (2012) [18], who discussed the behavior of a circular footing constructed on reinforced sand under the influence of some factors including number of geogrid layers under static and repeated loading. It is worth noting that less settlement observed when the other variables, such as footing depth, geogrid layers depth were remaining constant but the test velocity drop to 6 mm/sec.
B. Footing at depth

Figure 9. Settlement versus time for untreated and treated with 1 geogrid layer models with different depths and velocity=6 mm/sec

A. Footing at surface

B. Footing at depth

Figure 10. Settlement versus time for untreated and treated with 2 geogrid layer models with different depths and velocity = 9 mm/sec

A. Footing at surface

B. Footing at depth

Figure 11. Settlement versus time for untreated and treated with 2 geogrid layer models with different depths and velocity = 6 mm/sec
6.4. Model Test on Treated Soil with Geogrid and Fly Ash

Figures 12 to 15 present the typical relationship of settlements versus time for soil bed improvement with geogrid-fly ash layer. In general, the figures depict that the addition of fly ash in the presence of geogrid reduces the settlement compared to models without treatment. Even though models treated with geogrid or fly ash only. That is compatible with Tejaswini et al. (2014), who studied the performance of circular footing resting in reinforced fly ash beds under repeated loads [19]. Based on the results of experiments, the circular footing resting on reinforced fly ash beds performs better than its counterpart resting on unreinforced fly ash beds. Moreover, from these figures it can be noticed that the total settlement of a footing continues to increase during the time of the load and reaches a maximum value in most figures at the end of test. Also, it can be seen for models treated with 1 geogrid layer and fly ash that there is an improving in the settlement measured at the end of the test more than 40% for models footings placed at the surface (Df =0) and at the depth Df =100 mm with test velocity =9 mm/sec. In addition, a significant reduction in settlement value measured at the end of test for models treated with 2 geogrid layers and fly ash compared to untreated models. That is equal to 6.89 mm and 5.87mm for footing at Df=0 with velocity equal to 9 and 6 mm/sec respectively. A better performance occurs with inclusion of geogrid reinforcement that agrees with Choudhary et al. (2010) and Hotti et al. (2014) results [3, 6].

Figure 12. Settlement versus time for untreated and treated with 1 geogrid layer and fly ash models with different depths and velocity = 9 mm/sec

Figure 13. Settlement versus time for untreated and treated with 1 geogrid layer and fly ash models with different depths and velocity = 6 mm/sec
A. Footing at surface

B. Footing at depth

**Fig. 14:** Settlement versus time for untreated and treated with 2 geogrid layers and fly ash models with different depths and velocity = 9 mm/sec

**Fig. 15.** Settlement versus time for untreated and treated with 2 geogrid layers and fly ash models with different depths and velocity = 6 mm/sec

### 7. Conclusions

Depending on comprehensive laboratory tests performed on models soil with varying improvement techniques and untreated one, the following conclusions are drawn.

- In most models, an increase in settlement was measured at the end of test, which corresponds to the increase in test velocity from 6 to 9 mm/sec.
- In general, the settlement decreases with the increase in the footing depth for both test velocities and thus the bearing capacity increases with depth.
• The addition of fly ash causes a reduction in settlement of about more than 40 to 50% for footing at the surface and depth respectively.

• The settlement decreases slightly with reinforcing clay with 1 geogrid layer. While the settlement decreases remarkably with 2 geogrid layers for both footing placed at the surface or at depth.

• The addition of fly ash with geogrid (1 or 2 layers) reduces the settlement to about 50% for both velocities compared to those models treated with either geogrid only or fly ash only.

• The maximum settlement value is less for model treated with fly ash and two geogrid layers, which is equal to 5.87 mm at rate of velocity = 6 mm/sec, and equal to 6.89 mm at rate of velocity = 9 mm/sec compared with the other models.

8. Conflicts of Interest

The authors declare no conflict of interest.

9. References


Effective Utilization of Municipal Solid Waste as Substitute for Natural Resources in Cement Industry

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Abstract

The aim of this study was to evaluate the municipal solid waste (MSW) composition of Peshawar city and its affective utilization for energy purpose in the cement industry. A total 14 days consecutive testing of MSW samples was conducted for winter and summer periods for the purpose of evaluation of the waste composition followed by calculating its heating values. Compliance level of MSW at source was determined which was based upon the questionnaire distribution followed by the financial analysis and feasibility evaluation of the project. The results revealed that the average waste composition of the samples consists of organic waste contents (20.72%), combustible items (37.86%), readily saleable items (20.95%) and other miscellaneous waste items (20.46%). Moreover, the samples were then tested for the evaluation of calorific value and it was found that the heating value of MSW is recorded up to 35513 KJ/Kg whereas; the value for coal is around 38000 KJ/Kg. These findings revealed that the replacement of coal by MSW may be more efficient and might be effectively utilized in the production of cement as the energy production of MSW and coal is nearly same. In addition, the utilization of MSW as a replacement of coal has a great potential of enhancing the service life of the landfills. Besides, NPV analysis of this study revealed that the project is worthwhile to be implemented as it shows high returns regarding financial aspects.

Keywords: Municipal Solid Waste; Management Practices; Environmental Concerns; Dumping Areas; Calorific Value; Natural Resources.

1. Introduction

Currently, Pakistan is facing environmental problems and is under energy crisis. The country is also facing social concerns which are due to the mismanagement of MSW. These issues have achieved a greater level of alarming dimensions. Improper MSW management not only leads to environmental degradation but also attributes to the public health risks [1]. The enormous amount of waste being generated is due to the rapid industrialization, urbanization, over population, non-utilization of waste as a resource and improper MSW management from source to destination. The MSW generation rate is directly related to the socio-economic activities of the people. Higher social and economic activities lead to greater generation of MSW [2]. In urban areas, the expected population would be six billion people till 2050 [3, 4]. Prediction shows that the global population may reach up to nine billion by 2050 [3, 5]. Moreover, according to the assessment, the developing states may face 99% and 55% of population growth and urbanization rate respectively [3, 5, 6]. With the rapid pace of urbanization, the expected growth in rural and urban population may reach up to 34%
and 66% respectively [7, 8]. Till 2050, the decreasing trend of the rural population while enhancing the urban sprawl is shown in Figure 1.

Figure 1. Percentage showing rural and urban population globally [7, 8]

In comparison with the developed areas, the rural areas of the under developing regions are facing a decreasing trend in population whereas the urban areas are facing an increasing trend alarmingly. Pakistan like other developing states is also facing higher population growth rate. The current population of Pakistan is 189.12 million, out of which 115.2 million comprises of rural and 72.50 million of urban population respectively [9]. Currently, Pakistan is the sixth populated country in the world but its population is expected to reach to 363 million if it retains the same rate till 2050 [9]. Population growth rate (PGR) is a major indicator for the change in population that plays a vital role for a country to be economically developed. The PGR of Pakistan is more prominent in comparison with the Muslim and other neighboring states excluding Egypt and Afghanistan [9, 10]. People are drifting from rural to downtown areas in order to get better facilitation of education, jobs, life style and living and other such factors. This practice results in rapid urbanization growth in Pakistan. The population of metropolitan areas has increased from 38.6% to 39.2% during 2014 to 2015 [10], giving it a rating of three among the regional states as shown in Figure 2. Besides, the MSW generation rates in major cities of Pakistan in respective year is depicted in Figure 3 and tabulated in Tables 3 and 4.
Urban population will cross 122 million by 2030 if the current trend in urbanization is increasing continuously [8, 9, 11], that will ultimately enhance the energy consumption [12]. Energy source is directly related to the survival of growing population. Energy is taken as a lifeline of the economy and is the main indicator in the sustainability of commercial, domestic and industrial revolution activities. The primary energy consumption (PEC) in Pakistan in year 2014 increased at an annual rate of 3.6% that lead to 66.8 million tons oil equivalent (MTOE) with non-energy purpose of 3.4 MTOE and transformation of 23.6 MTOE [10, 13]. The entire primary energy consumption is estimated to be 69.21 MTOE in the year 2015 whereas 71.7 MTOE in the year 2016 as shown in Figure 4.

Pakistan is relying on non-renewable energy sources for the energy extraction whereas other countries are extracting energy from biomass [9, 14], as depicted in the Figure 5. Energy needs for Pakistan is anticipated to increase at an ACGR of 4.37% to 6.09% which is dependent on the GDP growth that ranges from 116-148 MTOE till 2022 as shown in Figure 6 [13, 15, 16].
The country is facing energy crises at an alarming accelerating rate and due to this fact; Pakistan is facing many challenges currently. Subsequently, this increasing breach amongst the energy supply and demand has given birth to economic crises in the country [17]. Due to the shortage of energy, the gas production is reduced that leads to dependency on fossil fuel and fewer usage of renewable resources like wind energy, solar energy or waste to energy programs [18, 19]. Moreover, the low budget situation of power supply firms, undeveloped power production capacity, limited hydel resources and cheap coal attributes to the energy shortage in the country [9, 12]. A financial development through an economic growth leads to positive and significant impact on energy consumption [12]. Besides, due to increasing population, the MSW generation is increasing each day that contaminates the environment and its surroundings by one way or another if the MSW is not managed in a right manner. Secondly, rapid urbanization is subjected to agricultural demand and industrial growth which ultimately requires energy sources. The world is using the non-renewable sources for energy production from centuries and the fossil fuel resources are depleting because of rapid growth in industrialization and population. Subsequently, the reduction of fossil fuel results in economic failure. Moreover, the energy utility pattern is overlapping specifically in Pakistan since the population and industrialization is growing at rapid rate [20]. As the income of Pakistan is categorized to be in the middle, the energy issues are getting worst since previous decades. Increase in economy and population leads to the continuous energy demands of the country [12]. Economic growth, development, energy security and sustainability are the key indicators of any energy system [10]. Due to shortage of power in the country, the economy’s long run growth has decreased from 6.5% to 2% [11, 12]. This parameter has negatively affected the exports, international competitiveness, employment and poverty in Pakistan [21].
It is the need of the current as well as future sustainability to save the world from environmental degradation and wastage of resources. The world is dependent upon fossil fuels since beginning. The reliance on such fossil fuel causes global warming that leads to the climate change issues due to human activities and utilization of these fossils [22]. These fossil fuels not only deplete the natural resources but also release various greenhouse gases i.e. CH\textsubscript{4}, CO\textsubscript{2} and NO\textsubscript{x} which causes adverse environmental impacts. The release of the mentioned gases from various sources has increased since 1970 that is mainly contributed by 40% from CH\textsubscript{4} and 60% from CO\textsubscript{2} [23]. The earlier studies [17, 24, 25], shows that MSW is releasing 550 Tg methane emissions per year globally which makes it the 4th largest contributor in the world. Some of the researches [26, 27], conclude that the mismanagement of MSW landfills contributed to third major source of methane emission which is predicted to increase to 816 MtCO\textsubscript{2}-eq till 2020 if they are not addressed in time. Currently, Pakistan is at place of 135th to contribute 0.8% of the global greenhouse gases and due to dependency on the non-renewable energy sources, this position is expected to increase by 2030 [17, 28, 29]. The large amounts of MSW generation due to technological development, economic expansion and changing lifestyle is attributing to serious environmental issues because of inappropriate management systems [30, 31, 32]. Like other developing states, Pakistan is also facing challenges in MSW Management because of the rapid urbanization and deficiency of formal recycling of waste into resources [28, 33, 34]. Despite of enormous amount of MSW generated, the economic growth of developing countries has severely affected due to the crises in energy [35]. Therefore, the discharge of emissions from plants that operates on non-renewable energy is one of the responsible parameters in environmental deterioration.

The world is looking up of substitutes for conventional non-renewable energy that would not only provide energy security but will also ensure an ecofriendly sustainable development. Likewise, the reliance on the conventional energy sources and improper MSW disposal leads to misbalance of demand and supply in the market that ultimately leads to the cost hypes of the non-renewable energy sources and over expenditures for handling the huge amount of MSW with no revenue generation. This practice is moving people to look up for more beneficial, viable and cheap energy sources across the globe [22, 35]. One the researchers [36] investigated the utilization of MSW as a replacement of alkali activated slag cement and concluded the improved characteristics of cement. Moreover, MSW is primarily used as a road subbase material, landfill structure material, embankment fill, as cement raw material and concrete products [37]. In one of the studies [38], MSW was successfully utilized in the production of eco-cement. This eco-cement was successfully produced from raw meals made up of 85% residues and 15% additives, and clinkered at 1000 °C, which was within the maximum operating temperature range of a typical incinerator. It showed no hydraulic reactivity but achieved an immediate compressive strength of over 50 MPa after carbonation activation.

In Pakistan, the exploitation of various renewable energy resources can never be denied, but unfortunately Pakistan is still under numerous energy crises and is facing a lot of challenges since last decades [12]. Hence, such energy crises may be counter balanced with the utilization of MSW. In this study, the MSW composition of Peshawar city along with its calorific value has been focused. In Pakistan, the annual progression rate of MSW generation is 2.4% that is replicating an alarming growth [28] and has raised a serious concern for its affective management [9, 28]. Concerning the mentioned issues, an effort has been made in this study to investigate the potential of MSW being generated in Peshawar city, Pakistan.

The rest of the paper is organized as follows. Section 2 briefly discusses the scope of the study, section 3 presents experimental program, section 4 provides results and discussion while the study is concluded in section 5.

2. Scope of the Study

In comparison with the developed states, the MSW generation rate in Pakistan is increasing day by day like other developing countries. The ability of implementing the 5R hierarchy of MSW management in its reduction, reusing, recycling, recovering and refusal is inadequate in the country. Consequently, the mismanagement of the MSW leads to economic, social and environmental issues. The MSW generation is directly proportional to the population growth. Since the population of Pakistan is increasing each day with the generation of huge amount of MSW that necessitates proper management but unfortunately, the MSW is thrown openly onto lands, water and alongside the streets and roads. Similar to other developing nations, Pakistan is not giving proper attention to the MSW management and the emerging concerns in this domain as a result of which the country is facing serious problems. The mismanagement of MSW is due to various reasons that include lack of human resources, improper infrastructure for MSW management facilities and weak financial conditions of the country. Therefore, this research work is carried out to find the MSW composition of Peshawar city, Pakistan followed by the assessment of calorific values of the combustible items for the purpose of energy generation along with monetary value of the saleable items in the MSW stream. In this way MSW would not only be utilized in an efficient way, rather to implement a sustainable MSW management program that might lead to the enhancement of economy, social acceptability and environmental feasibility of the country. For more details of the present study, the proposed plan for effective MSW management is depicted in Figure 8.
3. Experimental Program

3.1. Study Area

A pilot study was conducted in Peshawar city from December 2017 to June 2018. Peshawar is the capital of Khyber Pakhtunkhwa, Pakistan and is the ninth largest city of the country according to the census of 1998. It is situated in a valley near the Khyber Pass, eastern end. It is at a distance of around 170 Km from the capital of Pakistan. The climatic condition of the city is semi-arid, having slight winters and hot summers.

3.2. Waste Management Authority

Water and Sanitation Services Company Peshawar (WSSCP) is responsible for waste management of Peshawar city. WSSCP run under Peshawar Development Authority (PDA). The current waste management system focuses on collection, transportation and disposal only. PDA has divided the city into four major regions namely Town-I, Town-II, Town-III and Town-IV on the basis of collection regions whereas the disposal sites are located in three regions, i.e. Landi Akhun Ahmad dumping site, Hazar Khwani dumping site and Hayatabad Phase VII dumping site.

3.3. Dumping Sites

Hazar Khwani dumping site is owned by the government of KPK and is spread over an area of 1,089,000 sq-ft near Hazar Khwani chowk, Ring Road, Peshawar. The site is utilized for dumping the waste of town-I and II of Peshawar city that comprises of almost two-third region of the city. The site is situated at a distance of 6 km from ring road, adjacent to GT road bus terminal. The MSW is collected from the commercial hubs and urban areas including educational institutes, residential buildings and restaurants etc. There are 12 dumping pits and almost all of the pits have been occupied by the MSW till now. The area of each pit is around 54,500 sq-ft and the depth ranges from 100 to 300 ft. The area is surrounded by commercial and residential hubs.

Landi Akhoon Ahmad dumping site is utilized for dumping the MSW of town-III of Peshawar city that comprises of almost one-third region of the city. The site is situated at a distance of 1.5 km from ring road, adjacent to Pushakharah chowk. This site comprises of seven pits, out of these pits, only one pit is available for dumping that is being used till now. The area of each pit is around 63340 sq-ft whereas its depth ranges from 250 to 400 ft.

The MSW of town-IV is dumped in a site located in phase 7 Hayatabad, Peshawar. In this facility the MSW is dumped in the dumping pits followed by a layer of soil on the day end in order to mitigate the foul smell. This site is operational since 1988 but as this site is reserved for the MSW of Hayatabad Township only therefore, it has not yet reached up to its saturation level. Around 100 tons of MSW is dumped in this facility on daily basis. The MSW management system at this site is relatively in a better condition but still it didn’t reach up to the benchmark of an ideal MSW management system.

3.4. Categories of Society w.r.t. Income Levels

The composition of MSW greatly depends on the income levels of an area. The MSW generation from high class income levels will be having much more valuable items as compared to the MSW stream of a low class income level area. The classes of areas are divided in the following categories.

3.4.1. High Income Class

These are the residential areas having relatively good infrastructure like buildings, roads, water, and security. The high class income area is properly planned with good MSW management systems. The people of such area normally work in corporate sectors having good income levels. These areas are generally occupied with small family households.

3.4.2. Middle Income Class

These residential areas are characterized by small apartments or flats. These areas are normally occupied by relatively more households as compared to high income class areas. The residents of such an area are normally job oriented people with average income level.

3.4.3. Low Income Class

These are the areas with poor amenities and social services with unplanned infrastructure. Such areas are located on the peripheries of Metropolitan cities normally known as slum areas. The residents of such an area normally do petty jobs like working as foremen.

The current MSW management practices in Peshawar focuses only on the collection, conveyance and disposing off of the wastes without utilization of the valuable items from the MSW stream which eventually leads to an economic failure and an unsustainable system. The current and proposed plan of MSW management is depicted in Figures 7 and 8 respectively.

Figure 7. Current MSW management practices

Figure 8. Proposed MSW management practices

3.6. Sorting Procedure

For the testing process of municipal solid waste, the whole Peshawar city is divided into five zones i.e. Zone –I, Zone –II, Zone –III, Zone –IV, Zone –V. The testing method mainly comprised of door to door waste collection system, which was further followed by manual sorting technique. The testing method used for this purpose was ASTM D 5231-92. Bags were distributed in the targeted areas for the collection of the waste. The waste collecting bags were gathered from the selected household in the evening and brought to the site, where all the waste was mixed for homogeneity and then the total weight of the waste was recorded. Consequently, the samples were gathered in an 800 kg weight followed by conning and quartering of the bulked sample till a standard sample of 94 Kg to 136 Kg is achieved as shown in Figure 9. The sample was further segregated into various MSW items. After segregation process, each sample was weighed and its percentile was recorded as depicted in Figure 10. The testing method was employed for a consecutive period of
fourteen days in summer and winter seasons for the purpose of recording the variation and trends in the MSW stream. The sample of waste is categorized into various items as shown in the Table 1.

![Figure 9. Coning and quartering procedure of 800 kg sample](image)

![Figure 10. Sorting procedure of MSW](image)

<table>
<thead>
<tr>
<th>S. No</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
<th>10</th>
<th>11</th>
<th>12</th>
<th>13</th>
</tr>
</thead>
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<td>Particular</td>
<td>Bread</td>
<td>Paper</td>
<td>Plastic</td>
<td>Rubber</td>
<td>Glass</td>
<td>Metal</td>
<td>Food Waste</td>
<td>Yard Waste</td>
<td>Diapers</td>
<td>Leather</td>
<td>Textile</td>
<td>Ceramics</td>
<td>Debris</td>
</tr>
</tbody>
</table>

The percentile of each item was calculated by the following formula:

\[
\text{Percentage of each waste item} = \left( \frac{\text{Weight of segregated sample}}{\text{Weight of total sample}} \right) \times 100
\]

### 3.7. Heating Value of MSW

For calculating the calorific value of MSW items, standard methods of ASTM D 5468-02 was followed. Combustion is considered as one of the most significant parameters in the energy production domain. The combustion process implicates the oxidation of component in the fuel. The heat exploited through this reaction is known as heat of combustion. Such test is conducted in Anmol Laboratory, Lahore, Pakistan through bomb calorimeter apparatus. The sampling procedure for calculating calorific value of MSW is given as under.

The sample of MSW is dried in an oven followed by grinding it into a homogeneous powder using grinding machine. For the purpose of finding the calorific value of the sample, the powder is converted into lumps. Prior to taking the lump
for testing, calibration was performed using benzoic acid. After completing the calibration, MSW sample was suspended and the fuse wire was fastened across the electrodes followed by closing the oxygen bomb. The container of the calorimeter was filled with 2000 g of water whose temperature has been adjusted between 19-21°C. To reach the equilibrium, stirring was performed for around five minutes before starting a measured run. During the combustion process, the cell got heated, subsequently, the heat was transmitted to water placed in the cell and the temperature of the cell was recorded. After that, the bomb was fired by holding the ignition button till the light turned off. The reference lines for both initial and final temperatures were calculated as follows.

\[ \Delta T = (T_c - r(c - b)) - (T_a - r1(b - a)) = T_c - T_a - r1(a - b) - r2(c - b) \]

Gross calorific value is calculated by the following formula:

\[ GCV = \frac{(\Delta T_W - e1 - e2 - e3 - e4)}{m_s} \]

- \( W \): Energy equivalent of the calorimeter in °C identified by standard tests
- \( GCV \): Heat of combustion of mass sample in cal/g, gross calorific value
- \( m_s \): Mass (g);
- \( \Delta T \): Net temperature rise correction (°C);
- \( e1 \): Rectification of calorific values for heating of nitric acid when 0.0710N alkali utilized as a titrant;
- \( e2 \): Rectification of calorific values for heating of sulphuric acid;
- \( e3 \): Rectification of calorific values for combustion of fuse wire;
- \( e4 \): Rectification of calorific values for combustion of benzoic acid sample (6318 cal/gm) (mBA) cal;
- \( c1 \): Millimeters of standardized alkali solution;
- \( c2 \): Sulphur content in the sample (%);
- \( c3 \): Fuse wire consumed during firing (cm);
- \( W \): Energy correspondent of the calorimeter, determined in stabilization;
- \( m \): Mass (g).

### Table 2. Population and MSW generation rates of the targeted areas of Peshawar city, 2018

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Zones</th>
<th>Reach</th>
<th>Areas Covered</th>
<th>Income Level</th>
<th>Population (in capita)</th>
<th>No. of Average Households</th>
<th>Total Waste (Kg)</th>
<th>Total Houses covered</th>
<th>Total People generating waste</th>
<th>Sampling Duration (days)</th>
</tr>
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<tbody>
<tr>
<td>1</td>
<td>I</td>
<td>Charsadda Road link</td>
<td>Shahi Bagh Faqeer Abad Mahal Terai I Mahal Terai-II</td>
<td>Middle Income</td>
<td>93107</td>
<td>8</td>
<td>11480</td>
<td>190</td>
<td>1520</td>
<td>14</td>
</tr>
<tr>
<td>2</td>
<td>II</td>
<td>Ring Road Link</td>
<td>Sikandar Town Hazar Khwani Pishhtakara Achini Qamardin Garhi</td>
<td>Low Income</td>
<td>120917</td>
<td>8</td>
<td>10920</td>
<td>195</td>
<td>1560</td>
<td>14</td>
</tr>
<tr>
<td>3</td>
<td>III</td>
<td>University Road Link</td>
<td>Tehkal Arbab Road University Town Palosai Shaheen Town</td>
<td>High Income</td>
<td>129134</td>
<td>7</td>
<td>10360</td>
<td>210</td>
<td>1470</td>
<td>14</td>
</tr>
<tr>
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<td>IV</td>
<td>Old City Peshawar</td>
<td>Yakathooth Jahangir Pura Karim Pura Lahori Gate Gulbahar</td>
<td>Middle Income</td>
<td>137169</td>
<td>8</td>
<td>11060</td>
<td>200</td>
<td>1600</td>
<td>14</td>
</tr>
<tr>
<td>5</td>
<td>V</td>
<td>Hayatabad</td>
<td>Phase-I Phase-II Phase-III Phase-IV Phase-V Phase-VI Phase-VII</td>
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<td>34587</td>
<td>6</td>
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<td>1500</td>
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### Table 3. MSW generation rates in major cities of Pakistan during 2005, 2007 and 2010

<table>
<thead>
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<tr>
<td></td>
<td>Population</td>
<td>Generation Rate</td>
<td>Quantity (MT/year)</td>
<td>Population</td>
<td>Generation Rate</td>
<td>Quantity (MT/year)</td>
<td>Population</td>
<td>Generation Rate</td>
<td>Quantity (MT/year)</td>
</tr>
<tr>
<td></td>
<td>(million)</td>
<td>(kg/cap/day)</td>
<td></td>
<td>(million)</td>
<td>(kg/cap/day)</td>
<td></td>
<td>(million)</td>
<td>(kg/cap/day)</td>
<td></td>
</tr>
<tr>
<td>Karachi</td>
<td>10.82</td>
<td>0.61</td>
<td>2.42</td>
<td>0</td>
<td>0</td>
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### Table 4. MSW generation rates in major cities of Pakistan during 2012, 2013, 2014, 2016 and 2018 (Present) [45]

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Figure 11. Map of major cities of Pakistan
4. Results and Discussion

4.1. Waste Composition Analysis

The total MSW generation in Peshawar is around 900 tons/day. However, the average waste generation rate of the households from the collected areas comprises of around 0.51 kg/cap/day. The zone wise waste generation rates are 0.54, 0.5, 0.503, 0.493 and 0.54 kg/cap/day for zone-I, zone-II, zone-III, zone-IV and zone-V respectively, as shown in Table 2. The higher values of waste generated is attributed to the higher socio-economic activities of the residents living in that particular area. As discussed earlier about the different income levels, zone-I is relatively a middle income area having more number of households as compared to zone-V.

The testing was conducted in five mentioned zones of the city that was carried out for the winter and summer seasons (year 2017-18) for consecutive 14 days so as to observe the seasonal variation in the MSW composition. The timings for distribution of waste collection bags kept at 7:00 hours and its collection was done at 18:00 hours. The waste was then brought to the site for the purpose categorization as mentioned in sorting procedure. The waste composition of each day was recorded and analyzed in spread sheets. The MSW generation rates in major cities of Pakistan are shown in Tables 3 and 4 and graphically shown in Figure 11. While the MSW composition of winter and summer seasons are listed in Tables 5 and 6 respectively and graphically shown in Figure 12.

The results show that food waste has the highest values of 21.20% and 20.24% among all the waste items for winter and summer seasons respectively. This is due to the fact that food being the basic necessity of the human beings; therefore the food waste appeared in higher fractions. Moreover, the left overs of food items like tea bags, peels and other food related items which could not be further utilized are also included in the food waste. The second highest item in the MSW stream is diapers which accounts for 14.77% and 13.91% in winter and summer seasons respectively. Since, the trend of using disposable diapers and wipes is more common rather to use normal washable clothes because of the changing lifestyles and urbanization, therefore it is being utilized in a very huge quantity and such diapers and wipes eventually joins the MSW stream. Paper waste is the third highest waste generated item and its values ranges to 14.54% and 13.75% for winter and summer seasons respectively. As the corporate sectors in the metropolitan and downtown areas of the city demands for higher printing activities to fulfill their requirements therefore, the generation of paper waste is a major concern. Plastic items are being used very commonly for packing different edible items such as cold drinks, juices and meals while their contribution is up to 13.80% and 12.20% in the winter and summer seasons respectively. Whereas, the glass item was used to be in greater quantities for packing the edible items like tea, coffee, juices and meals while their contribution is up to 13.80% and 12.20% in the winter and summer seasons respectively which is more reasonable generation. Ceramics are normally produced from construction projects and houses as a waste, since such waste is produced only when it gets broken down during handling, therefore, its quantity hardly reaches to 2% of the MSW stream.

It is revealed that there is not much variation between the results of winter and summer MSW compositions except that the yard waste which demonstrates that during winters, the yard clipping activities drastically decreases as the production of the grasses in yards during such seasons is negligible in comparison with summer season. Moreover, the mean MSW composition of the winter and summer seasons along with various purposes for which the waste might be utilized is shown in Figure 13. While the dumping and utilization assessment of the current and proposed plan is depicted in Figure 14.

4.2. Overall Variation of MSW Throughout the Period

In overall testing period, the bread component, plastic, rubber, food waste, yard waste, diapers, debris in the waste stream was in increasing trend while paper, glass, metal, leather, textile and ceramics were in decreasing trend.

Table 5. MSW composition of winter season

<table>
<thead>
<tr>
<th>Date</th>
<th>Bread</th>
<th>Paper</th>
<th>Plastic</th>
<th>Rubber</th>
<th>Glass</th>
<th>Metal</th>
<th>Food waste</th>
<th>Yard waste</th>
<th>Diapers</th>
<th>Leather</th>
<th>Textile</th>
<th>Ceramics</th>
<th>Debris</th>
</tr>
</thead>
<tbody>
<tr>
<td>22/12</td>
<td>9.96</td>
<td>18.76</td>
<td>11.44</td>
<td>2.89</td>
<td>7.12</td>
<td>4.18</td>
<td>17.6</td>
<td>3.17</td>
<td>5.54</td>
<td>3.51</td>
<td>4.09</td>
<td>1.79</td>
<td>0.63</td>
</tr>
<tr>
<td>23/12</td>
<td>10.73</td>
<td>17.35</td>
<td>9.66</td>
<td>2.62</td>
<td>5.94</td>
<td>4.28</td>
<td>20.33</td>
<td>4.56</td>
<td>11.43</td>
<td>4.04</td>
<td>4.12</td>
<td>1.59</td>
<td>0.54</td>
</tr>
<tr>
<td>24/12</td>
<td>13.98</td>
<td>16.92</td>
<td>13.48</td>
<td>5.24</td>
<td>2.56</td>
<td>4.5</td>
<td>21.89</td>
<td>1.73</td>
<td>16.11</td>
<td>2.69</td>
<td>2.61</td>
<td>2.3</td>
<td>0.44</td>
</tr>
<tr>
<td>25/12</td>
<td>12</td>
<td>13.76</td>
<td>13.27</td>
<td>2.03</td>
<td>5.66</td>
<td>5.66</td>
<td>22.21</td>
<td>1.34</td>
<td>15.41</td>
<td>3.97</td>
<td>3.3</td>
<td>1.12</td>
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</tr>
<tr>
<td>26/12</td>
<td>9.7</td>
<td>12.2</td>
<td>15.97</td>
<td>2.55</td>
<td>6.16</td>
<td>3.57</td>
<td>20.16</td>
<td>3.59</td>
<td>16.09</td>
<td>3.59</td>
<td>4.11</td>
<td>1.23</td>
<td>0.56</td>
</tr>
<tr>
<td>27/12</td>
<td>15.37</td>
<td>14</td>
<td>18.7</td>
<td>3.63</td>
<td>4.51</td>
<td>2.95</td>
<td>20.73</td>
<td>1.72</td>
<td>13.19</td>
<td>3.29</td>
<td>2.36</td>
<td>2.32</td>
<td>0.86</td>
</tr>
</tbody>
</table>
Table 6. MSW composition of summer season

<table>
<thead>
<tr>
<th>Date</th>
<th>Bread</th>
<th>Paper</th>
<th>Plastic</th>
<th>Paperboard</th>
<th>Glass</th>
<th>Metal</th>
<th>Food waste</th>
<th>Yard waste</th>
<th>Paper</th>
<th>Diapers</th>
<th>Leather</th>
<th>Textile</th>
<th>Ceramics</th>
<th>Debris</th>
</tr>
</thead>
<tbody>
<tr>
<td>07/06</td>
<td>10.3</td>
<td>18.23</td>
<td>12.51</td>
<td>2.65</td>
<td>6.96</td>
<td>4.12</td>
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<td>6.43</td>
<td>11.34</td>
<td>4.28</td>
<td>4.19</td>
<td>1.98</td>
<td>0.58</td>
<td></td>
</tr>
<tr>
<td>08/06</td>
<td>10.13</td>
<td>15.27</td>
<td>11.37</td>
<td>3.87</td>
<td>5.54</td>
<td>3.97</td>
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<td>6.78</td>
<td>15.5</td>
<td>3.87</td>
<td>6.17</td>
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<td>09/06</td>
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<td>3.33</td>
<td>4.08</td>
<td>19.12</td>
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<td>12.43</td>
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<td>2.59</td>
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<tr>
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<td></td>
</tr>
<tr>
<td>11/06</td>
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<td>5.69</td>
<td>11.23</td>
<td>3.12</td>
<td>3.64</td>
<td>1.83</td>
<td>0.57</td>
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<tr>
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<td>12.75</td>
<td>14.3</td>
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<td>4.98</td>
<td>14.99</td>
<td>4.05</td>
<td>3.54</td>
<td>1.89</td>
<td>0.67</td>
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<tr>
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<td>12.98</td>
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<td>1.76</td>
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</tr>
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<td>5.23</td>
<td>4.98</td>
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<td>6.59</td>
<td>10.09</td>
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<td>3.96</td>
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<tr>
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<td>5.35</td>
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<td>20.2</td>
<td>6.7</td>
<td>11.82</td>
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<td>3.72</td>
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<td>0.77</td>
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</tr>
<tr>
<td>16/06</td>
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<td>14.19</td>
<td>12.04</td>
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<td>5.76</td>
<td>16.17</td>
<td>2.97</td>
<td>3.14</td>
<td>1.21</td>
<td>0.75</td>
<td></td>
</tr>
<tr>
<td>17/06</td>
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<td>12.11</td>
<td>11.98</td>
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<td>5.26</td>
<td>3.98</td>
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<td>15.5</td>
<td>1.9</td>
<td>2.4</td>
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<tr>
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<td>10.93</td>
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<td>5.23</td>
<td>3.33</td>
<td>20.24</td>
<td>6.74</td>
<td>12.33</td>
<td>3.68</td>
<td>3.79</td>
<td>0.93</td>
<td>0.4</td>
<td></td>
</tr>
<tr>
<td>19/06</td>
<td>13.23</td>
<td>14.56</td>
<td>12.12</td>
<td>3.23</td>
<td>7.26</td>
<td>2.56</td>
<td>20.66</td>
<td>5.34</td>
<td>16.14</td>
<td>3.77</td>
<td>2.34</td>
<td>2.54</td>
<td>1.23</td>
<td></td>
</tr>
<tr>
<td>20/06</td>
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<td>12.45</td>
<td>9.4</td>
<td>2.89</td>
<td>6.82</td>
<td>3.81</td>
<td>19.17</td>
<td>6.11</td>
<td>15.35</td>
<td>3.28</td>
<td>3.44</td>
<td>1.94</td>
<td>0.63</td>
<td></td>
</tr>
</tbody>
</table>

Ave: 12.12 13.75 12.20 3.21 5.44 3.89 20.24 5.86 13.91 3.40 3.43 1.86 0.68

Figure 12. Average municipal solid waste composition (Dec 2017 to June 2018)
4.3. Compliance Level of MSW at Source Segregation

A survey of source separation of waste into organic, combustible, readily saleable and other waste items was conducted in the targeted area which was based upon the questionnaire distribution. Out of 2600 respondents, 2127 (81.8%) were willing to go for a source segregation system, 255 (9.8%) were unwilling for this practice whereas 218 (8.4%) did not responded to the questionnaire. The main reason for willingness attributes is due to the fact that the respondents well understand that there is lack of proper MSW management and consequently a little effort of the
respondents can lead to a cleaner society and a better MSW management system. On the other hand, the unwillingness of some of the respondents attributes to the fact that they lack confidence in changing the system. Whereas, the respondents excuses to respond is because they were unaware of the worth of proper MSW management practices.

4.4. Financial Analysis

The total daily generation of MSW in Peshawar city is 900 tons. To assess the tonnage of each MSW item, the waste composition acquired from the testing is performed on 900 tons. Furthermore, a detailed survey regarding the monetary value of the saleable items was also conducted at the scrap purchasing vendors situated near Jamrod road, Peshawar (Industrial Estate, Hayatabad). The saleable items are sold out by the scavengers to the scrap dealers which are further purchased by the local material contractors for supplying it to the respective industries.

Table 7 shows the quantity in tons of the saleable items generated on a daily basis. The tonnage is based on the total waste estimates of the city. Subsequently, the total average revenue generation of the saleable items is acquired from the MSW stream. The total tonnage of the combustible items on a daily basis is shown in Table 8.

<table>
<thead>
<tr>
<th>Components</th>
<th>Average sample (Kg)</th>
<th>Expected daily generation (900 tons)</th>
<th>Price per ton (USD)</th>
<th>Revenue generation/day (USD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bread</td>
<td>11.96</td>
<td>108</td>
<td>120</td>
<td>12960</td>
</tr>
<tr>
<td>Glass</td>
<td>5.22</td>
<td>47</td>
<td>15</td>
<td>705</td>
</tr>
<tr>
<td>Metal</td>
<td>3.75</td>
<td>46.98</td>
<td>190</td>
<td>8926</td>
</tr>
<tr>
<td>Total daily revenue</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>22591</td>
</tr>
<tr>
<td>Total monthly revenue</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>677730</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Components</th>
<th>Average Sample (Kg)</th>
<th>Expected Daily Generation (900 Tons)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Paper</td>
<td>14.15</td>
<td>127.35</td>
</tr>
<tr>
<td>Plastic</td>
<td>13</td>
<td>117</td>
</tr>
<tr>
<td>Rubber</td>
<td>3.15</td>
<td>28.35</td>
</tr>
<tr>
<td>Yard</td>
<td>4.25</td>
<td>38.25</td>
</tr>
<tr>
<td>Textile</td>
<td>3.31</td>
<td>29.79</td>
</tr>
</tbody>
</table>

4.5. Feasibility Evaluation of project

4.5.1. Evaluation for the Combustible Items

Calorific values of MSW items

The MSW items specified for evaluating calorific values were tested in the Anmol Scientific Laboratory, Lahore, Pakistan. All of the tests were conducted at room temperature ranging from 19°C to 21°C. Both separate and mixed samples of waste were analyzed during the testing operations. The heating values and comparative cost analysis of MSW and coal are listed in Tables 9 and 10 respectively.

<table>
<thead>
<tr>
<th>S. No.</th>
<th>MSW item</th>
<th>Calorific Value (KJ/Kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Mixed MSW</td>
<td>35513</td>
</tr>
<tr>
<td>2</td>
<td>Yard waste</td>
<td>18158</td>
</tr>
<tr>
<td>3</td>
<td>Plastic</td>
<td>28970</td>
</tr>
<tr>
<td>4</td>
<td>Textile</td>
<td>18079</td>
</tr>
<tr>
<td>5</td>
<td>Rubber</td>
<td>8171</td>
</tr>
<tr>
<td>6</td>
<td>Paper</td>
<td>13560</td>
</tr>
</tbody>
</table>
The heating value of coal is around 30000 KJ/Kg to 38000 KJ/Kg. The coal being utilized in Cherat cement factory, Khyber Pakhtunkhwa, Pakistan during the incineration activity of manufacturing process of cement that costs around 310 USD per ton. Meanwhile, the same quantity of MSW costs 110 USD per ton. However, the MSW if utilized for the same purpose might induce a huge price cut off in the final product. An analysis has been performed on the economics between the two items i.e. coal and MSW. Table 11 shows a comparison of the cost of the cement unit price when coal and MSW is utilized as a combustible material.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Cost of coal per bag (USD)</th>
<th>Cost of MSW per bag (USD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cost of combustible items</td>
<td>1.12</td>
<td>0.059</td>
</tr>
<tr>
<td>Material cost</td>
<td>1.49</td>
<td>1.49</td>
</tr>
<tr>
<td>Government tax</td>
<td>1.12</td>
<td>1.12</td>
</tr>
<tr>
<td>Total cost of final product</td>
<td>3.73</td>
<td>2.67</td>
</tr>
</tbody>
</table>

From the above parametric study, one can easily assess the financial feasibility of the conventional and proposed plan of BCR (benefit to cost ratio).

\[

cr = \frac{\text{Value of benefits}}{\text{Value of costs}} = \frac{\text{Unit price of cement per bag by coal}}{\text{Unit price of cement per bag by MSW}}
\]

BCR = 3.73/ 2.67
BCR = 1.39

Thence, the proposed plan is feasible. Feasibility of a project is ascertained with the help of BCR. BCR corresponds to a conventional way of analyzing cost versus benefits that is performed for the purpose of finalizing the entire feasibility of the project. For its value equal or greater than 1, the project is considered feasible. However, for a project the value of BCR if less than 1, it cannot be implemented [46].

4.5.2. Evaluation for the Readily Saleable Items

In order to initiate this work, labor is needed to segregate this waste so as to provide a quick segregation process. A detailed survey has been conducted near the Khyber Teaching Hospital Peshawar, wherein the labor were interviewed for the daily wage they are getting. 40 labors were interviewed for this purpose. Among these 40 labors, 30 were working for a daily wage of 4.75 USD whereas the remaining 10 labor were willing to do the work for 4.2 USD. According to the survey done in the field, it takes around one hour for a person to segregate 100 kg of waste. Table 12 shows the monthly labor expenditures required to segregate 900 ton of daily waste generated in Peshawar city.

<table>
<thead>
<tr>
<th>Waste Generated (Tons)</th>
<th>900</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shift (Hrs.)</td>
<td>8</td>
</tr>
<tr>
<td>Labour (Quantity)</td>
<td>1100</td>
</tr>
<tr>
<td>Wage (USD)</td>
<td>4.5 (Avg.)</td>
</tr>
<tr>
<td>Daily Expenditures (USD)</td>
<td>4950</td>
</tr>
<tr>
<td>20% Overhead (USD)</td>
<td>990</td>
</tr>
<tr>
<td>Daily Expenditures (USD)</td>
<td>5940</td>
</tr>
<tr>
<td>Monthly Expenditures (USD)</td>
<td>178200</td>
</tr>
</tbody>
</table>
Water and Sanitation Services Company, Peshawar (WSSCP) works under PDA and WSSCP provides services for the MSW management in Peshawar City. The detail of monthly expenditures of WSSCP for management of the MSW along with total expected labor cost is depicted in Table 13.

Table 13. Current expenditures by WSSCP for MSW management along with expected labor cost

<table>
<thead>
<tr>
<th>Operations</th>
<th>Costs (USD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>O &amp; M Costs</td>
<td>3800</td>
</tr>
<tr>
<td>Staff Salaries</td>
<td>31200</td>
</tr>
<tr>
<td>Labour Costs</td>
<td>263250</td>
</tr>
<tr>
<td>Grand Total (Monthly)</td>
<td>298250</td>
</tr>
</tbody>
</table>

Furthermore, a financial assessment of the expenditures and expected revenue generation from the saleable items of the MSW has been performed to find out the BCR and feasibility of the proposed plan. In this concern, the formula of BCR is stated as:

$$\text{BCR} = \frac{\text{Generation of Revenue}}{\text{Total cost for MSW management}}$$

$$\text{BCR} = \frac{677730}{178200}$$

$$\text{BCR} = 3.80$$

![Figure 15. Net inflows and outflows (USD)](image1)

![Figure 16. Net inflows and outflows (USD)](image2)
The BCR value clearly states that the project is feasible since the value of BCR is above one. A 3.80 BCR depicts that by investing 1 USD, the return is 3.80 USD. The result reveals a considerable revenue generation potential from the MSW of Peshawar city.

The Figures 15 and 16 show the net inflows and outflows of the project for a period of 10 months. According to the field of economics and finance, a project is feasible if the Net Present Value (NPV) is positive. It can be seen that the NPV of the project is 0.85 million USD. NPV is the net present value of a cash flow. As the money that is to be received after 2-3 years does not have the same value as it has today. So, from the NPV value it is evident that the money which will come after 1-10 months from now is worth waiting for whilst we spend our investment today. Since the NPV for the project is not only positive but it shows high returns. Hence from a finance perspective, this would be an ideal project.

As the average budget of PDA is 72 million USD per annum (PDA Budget, 2015-2016), this project is not only self-sustainable, but will also account for 11.29% of PDA’s annual budget. This will be better than any other sources of budget for PDA itself as PDA would not be reliant on third-party customers or government for sanctioning this budget.

5. Conclusions

- From the findings it is revealed that the municipal solid waste (MSW) collected from Peshawar city holds huge impedence on energy production. For instance, the expected energy production of both the MSW and coal is nearly equal hence; the substitution of coal by MSW proved to be more efficient and might be effectively utilized for the production of cement. Consequently, the cement industry would not rely on the natural energy resources that lead to the utilization of MSW for its production.
- The data revealed that by the utilization of MSW as an energy resource induced a price cut off up to 28.4% per bag of cement production.
- The MSW being utilized for the energy production has a great potential of enhancing the service life of the landfills, subsequently the environment may be avoided by further deterioration. Since, the landfills located in Peshawar city are experiencing a huge amount of waste on daily basis; the utilization of MSW in the production of cement might enhance the capacity of dumping sites by 37.87% as it would not get dumped into the landfills. The residents living in the surrounding of these landfills can be secured from the unhygienic conditions and health issues associated with open dumping.
- Currently, PDA is spending 35000 USD on collection, transportation and dumping of the city waste and getting no return on the investment. However, the revenue generation from the saleable items if implemented at the authority level will not only generate monthly revenue of 6,77,730 USD but it will enhance the capacity of the dumping site by 20.95% as well.
- A benefit to cost ratio (BCR) of 1.39 demonstrates that the project might be feasible, economical and may be effectively implemented in cement industry. Consequently, the inflation in cement may be controlled and hence it may flourish the construction industry by encouraging the investors. On the other hand, the saleable items alone produce a BCR of 3.80 which will not only induce high returns into the system but will create employment opportunities for the individuals as well. Moreover, by utilizing the waste as a useful resource, a greater execution of waste management regulations with an improved relationship of public authorities with private sectors might be established.
- An NPV of 0.85 million USD shows that the project is worthwhile to be implemented as it shows high returns regarding financial aspects. Meanwhile, the local government of Pakistan is responsible for sanctioning developmental funds to Peshawar Development Authority (PDA). The annual budget of PDA for the year 2015-2016 is 72 million USD for such practices. By implementation of this proposed study, PDA might not only be able to manage the MSW effectively, rather it would induce a cash inflow of 11.29% to its annual budget, hence PDA might not be dependent on a local government for sanctioning of funds up to some extent (11.29%) that may lead to a self-sustainable system of the authority.
- The organic wastes if utilized for the production of animal feed or compost fertilizers may enhance the serviceability of the landfills by 20.72%. However, the other leftover MSW which comprises of only 20.47% of the total MSW stream that will eventually be dumped in the landfill which is quite lesser in comparison with the current MSW management practices. The significance of the data acquired by the authors suggest further study on sanitary landfilling technique rather than open dumping as abundant energy might be expected from biogas generation.

6. Acknowledgement

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7. Conflicts of Interest
The authors declare no conflict of interest.

8. References


Laboratory Investigation on Interaction of the Pile Foundation Strengthening System with the Rebuilt Solid Pile-Slab Foundation

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Abstract

The article presents the results of laboratory studies of pile model foundations in order to determine the effectiveness of the method proposed by the authors for its reconstruction in pile-tile foundation with preliminary pressing and cementation of the soil base. The studies were carried out on small-scale wooden models of foundations in the conditions of a soil paste. The models of foundations were subjected to vertical static loading in a laboratory tray with a diametrical transparent wall. The program of experiments was provided for determination of precipitation of the models: pile foundations without strengthening, with strengthening in the form of reconstruction from the combined foundation and with strengthening in the form of reconstruction into the combined foundation with preliminary stress of the soil base in the span part. Vertical and horizontal movements in the soil mass were also recorded by a contactless method (PIV) in every stage of model loading. On the basis of experimental measurements digital processing of data of sediments and displacements is performed, for drawing plots of sediments, epures and isolines of displacements in the soil base. The main result of the research is confirmation of the high efficiency of the proposed method of strengthening pile foundations due to the maximum use of pre-pressed soil base resources in spans between pile rows. It has been found that compression (pre-stress) significantly reduces soil deformability and allows to include it in operation without additional deformations. The use of pre-compaction reduced the precipitation of the model combined foundation by almost 40%, relative to the combined without compaction. The results of the research open the possibility to develop new methods of strengthening pile foundations from the point of view of effective inclusion in the operation of the soil base in the span part, due to its preliminary tension.

Keywords: Strengthening the Buildings; Soil Deformations; Soil Bed; Pile Foundation; Combined Pile-Slab Foundations (CPSF).

1. Introduction

Rational use of financial and material resources through renewal of residential buildings and urban development makes it possible to save the housing facilities and increase the usable floor area by reconstructing the facilities (40-70%). It is necessary to enumerate the main reasons which cause the need to strengthen the foundations and harden the foundation soils. The following classification is used in this research (Pronozin et al. 2019 [1], Utenov et al. 2017 [2], Khomyakov et al. 2017 [3]):

1.1. Reconstruction of Buildings

Reconstruction of buildings including major repairs, adding storeys, and increased foundation loading.

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Reconstruction and major repairs of buildings and structures are carried out in order to eliminate their physical deterioration and provide their expansion or additional functional use; this is accompanied by strengthening or replacing the structural elements of the building, adding storeys, deepening basements and making basement storeys, interior redevelopment and modification. The result is as follows: foundation loading increases by 30 to 50% (Khomyakov et al. 2017 [3], Mali and Singh 2018 [4], Samorodov 2018 [5], Stepanov et al. 2018 [6]). Thus, pile foundations suffer from overloading and this results in the need to strengthen them by embedding extra piles or strengthening the soil bed.

1.2. Deterioration of Foundations

It is necessary to point out two basic groups of reasons which cause deterioration of foundations.

Firstly, physical and mechanical processes result from interaction of foundations with the environment, namely: wooden members of foundations decay, the binding material is leached, and the masonry is destructed in an aggressive environment; corrosion of the reinforcement, frost destruction, etc.

Secondly, mechanical reasons are caused by various external effects (earthworks in the vicinity of the buildings, dynamic effects of transport and building machinery, etc.). In here, destruction is characterized by masonry disintegration and crumbling of mortar from the joints, cracks in the concrete and reinforced concrete foundations resulting in the loss of strength and rigidity of the foundations. Foundations and lower sections of the walls are destructed as a result of violated water isolation of perimeter walks around the buildings due to fluctuations in the groundwater level (Polishchuk and Tarasov 2017 [7], Stepanov and Rybak 2019 [8], Pronozin et al. 2019 [9]).

1.3. Changed Properties of Foundation Soils

During operation of the building to be restored, reconstructed or repaired, it is possible to change the hydrogeological situation within the core area of the soil bed. For example, extra moistening leads to deterioration of the physical and mechanical properties of soils; the strength of the bed decreases and its deformability increases (Shamsi et al. 2019 [10], Lee et al. 2018 [11], Ihsan and Toma Sabbagh 2017 [12], Rabiei and Choobbasti 2015 [13], Pronozin et al. 2018, 2019 [14, 1]).

1.4. Unacceptable General or Local Deformations of the Buildings

Unacceptable general or local deformations of the buildings are usually caused by errors in engineering geological surveys, design and construction of the soil beds and foundations of buildings, their operation, and construction of buildings and structures in the vicinity of the existing ones. In this case, significant non-uniform deformations lead to defects in the superstructures and accidental consequences.

One of the possible effective ways to increase reliability and reduce deformability of the pile-strip foundations is to strengthen them by rebuilding to continuous combined pile-slab or pile-shell foundations by crimping the soil bed. In the span it is achieved by pumping the pressurized mortar mix under the foundation foot (Pronozin et al. 2019 [14], Mali and Singh 2019 [15], Stepanov et al. 2018 [6], Kumar et al. 2017 [16], Mangushev and Nikiforova 2017 [17]).

2. Experimental Laboratory Investigation

The laboratory investigation aimed at the following:

Studying the interaction of the combined pile-slab foundations (CPSF) with a crimped soil bed, i.e. a qualitative assessment of the influence of the foundation system parameters on the deformed soil bed.

A single pile (model No.1), a slab foundation (model No.2), a pile-slab foundation (model No.3) and a pile-slab foundation with the crimped soil bed (model No.4) were considered as foundations.

The experiments included the following:

- Static tests conducted on the foundation models;
- Deformation fields (total, vertical and horizontal) determined in the soil bed given different models of foundations;
- Study of the clayey bed deformability during its loading;

The tests were carried out in an experimental tray which was filled with the clayey paste of a disturbed structure.

A metal half-cylindrical container with a front transparent wall of 1 m in diameter and 0.8 m in height was used as the experimental tray (Figure 1). The transparent wall was made of Plexiglas of 10 mm in thickness. A levered rig was used for loading which increased multiply to 4.2. Three independent frames served for measuring, lighting and photographing equipment to be installed.
Figure 1. Configuration of the experimental rig

The model of the pressed-in pile was made of a wooden cylinder of 25 mm in diameter and 250 mm in length sawn longitudinally. The model of the slab foundation was made of a particle board, 300×300×20 (h) mm in dimensions.

The model of the pile-slab foundation (Figure 2b) was like the model of the slab foundation made of the particle board 300×300×20 (h) mm in dimensions and six piles made of wooden cylinders of 25 mm in diameter, 250 mm in length and 15 cm in pile spacing.

The model of the pile-slab foundation with the crimped soil bed was similar to the model of the pile-slab foundation with the layer of a rubber shell in the contact zone of the slab spans (Figure 2c). The shell presented an expanding rubber chamber with a nipple connected to the compressor. The slab contact surface together with the chamber used for crimping was 80% of the total area of the slab. Air was pumped under the span in the shell for 1-2 min at a pressure of $p_{\text{crimping}} = 0.45$ atm (45 kPa). The pressure was monitored using pressure gauges mounted on the compressor.

A specially prepared soil paste with given physical and mechanical characteristics was used in the experiments: density $\rho=1.9$ g/cm$^3$, porosity coefficient $e = 0.7$, water content $w = 22.7\%$, plasticity index $I_p = 8.7$, liquidity index $I_L = 0.60$, angle of internal friction $\phi_0 = 15$, specific cohesion $c = 24.9$ kPa, modulus of deformation $E = 6 - 7 \text{ MPa}$.

Figure 2. Models of the foundations under study, a) single pile (model No.1), b) slab foundation (model No.2), c) pile-slab foundation (model No.3), d) a pile-slab foundation with the crimped soil bed (model No.4); 1 - pile $\varnothing 25$ mm; 2 - slab 300×300×20; 3 - mortar injection area; 4 - grillage.
When the soil had been compacted, the transparent side wall was demounted in order to install the marks. The marks were installed on a square grid of 0.02×0.02 m in dimensions using a template. The marks were made of a cylindrical polymer tube of 2 mm in external diameter, 1 mm in internal diameter and 5 mm in length.

In the case of static testing, the foundation models were loaded by means of metal loads through the levered rig; the value of loading was regulated by the dead load mass. Each loading stage was maintained until conditional stabilization occurred, when the rate of the settlement did not exceed 0.1 mm in the last 4 hours of observations.

The piles were immersed by means of the levered rig as well. The settlements of the foundation models were recorded using Aistov 6 PAO inclinometers with a scale value of 0.01 mm. To obtain more accurate experimental data, each series of experiments was conducted at least three times to control the repeatability of the results obtained.

3. Results and Discussion

In order to study the deformability of the soil bed under loading by different foundation models, the functions of the settlement versus the average pressure under the foundation foot were obtained (Figure 3).

The functions of the settlement versus loading for slab and pile-slab foundation models up to the third stage of loading were characterized by initial linearity. Nonlinear deformations appeared if pressure values increased.

The settlements of the models No.3 and No.4 were equal prior to crimping, up to the fifth stage, corresponding to \( N = 5kN \) (\( p_{\text{aver}} = 55 \text{ kPa} \)).

After the fifth stage air was forced under the pressure of \( p_{\text{crimp}} = 45 \text{ kPa} \) under the footing of the model No. 4 in the expanding rubber chamber to simulate the process of soil bed crimping.

Deformations of the soil bed and its compaction occurred when pressure was forced under the slab span, and the settlement did not increase. This is some kind of the analogue of the pre-stressed structural members, e.g. reinforced concrete structures.

The soil bed having been crimped and the load on the model No.4 having been increased, the rate of its settlement significantly decreased in relation to the similar model No. 3, but without any crimping. In here, when extra loading had been applied corresponding to an average pressure of 88 kPa, the settlement of the model No. 3 was 16.5 mm, model No. 4 - 11 mm, i.e., 1.5 times less.

At the last general stage of loading corresponding to \( p_{\text{aver}} = 111 \text{ kPa} \), the settlement of the model No.2 was 29.5 mm, model No.3 – 23.3 mm, model No.4 – 14.7 mm.

Thus, the final settlements of the model No.4 proved to be 37% less than those of the model No.3 due to crimping of the soil bed in the contact layer. It is important to underline that the crimping pressure was 40% of the average pressure (\( p_{\text{aver}} = 111 \text{ kPa} \)); this correlated with the difference of the final settlements.

![Figure 3. Settlements of foundation models versus vertical pressure values](image-url)
In order to determine deformations of the soil bed, an axisymmetric OZ coordinate frame was introduced, where Z axis passed through the center of the stamp and was directed vertically down, the axis passed through the center point of the stamp foot and was directed horizontally to the right; accordingly, the contact point of the model and the foundation soil along the axis of its symmetry was the center of coordinates.

Vertical, horizontal and total deformations of the soil were to be determined during the experiment. Soil deformations for pile-slab foundations were evaluated as the difference between the changed geometric position of the marks in the OZ plane and their initial location.

Using this technique, the contour curves of the whole desired deformations were plotted for the foundation models No.3 and No.4 for the stages at \( N = 5kN \) (\( p_{\text{average}} = 55 \) kPa) and \( N = 10 kN \) (\( p_{\text{average}} = 111 \) kPa).

Thus, at \( N = 5 kN \) (\( p_{\text{average}} = 55 \) kPa) the maximum values of the vertical soil displacements under the contact surface of the foundation model No.3 were 14 mm in the span. The deformations extended to a depth of \( 1.0B \) (where \( B \) is the width of the foundation), and the maximum values of the vertical soil displacements under the contact surface were 17 mm in the span for the model No.4 after crimping. Deformations extended to a depth of \( 1.1B \).

When the load increased, the maximum values of the vertical displacements of soil under the contact surface of the foundation model No.3 were 21 mm in the span, at \( N = 10 kN \) (\( p_{\text{average}} = 111 \) kPa) (Figure 4).

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**Figure 4.** Curve of vertical displacements of soil in the pile space for the foundation model No.3 and foundation model No.4, with \( N = 10 kN \) (\( p_{\text{average}} = 111 \) kPa): a - vertical \( w (z) \), b - total \( \Delta (z, x) \)
Deformations extended to a depth of $1.7VB$ ($B$ is the width of the foundation). The maximum values of the vertical soil displacements in the span were 19 mm for the foundation model No.4 ($p_{\text{average}}=45\text{kPa}$ – crimping, $p_{\text{average}}=111\text{kPa}$ - stage). Deformations extended to a depth of $1.3B$. The depth of the compressible thickness decreased due to compaction of the soil in the upper part of the core and unloading of the piles.

4. Conclusions
The laboratory investigation resulted in the following:

- Crimping of the soil bed performed on small-scale models in the span of the foundation at a rate of 40% of the total final load made it possible to reduce the foundation settlement by 37%.
- After crimping, the settlement was halved as compared to the settlement of the same foundation without crimping of the soil bed.
- The crimped soil bed made it possible to reduce compressible thickness by 23% as compared to similar foundations being loaded without any crimping.
- The crimped soil bed in the span of pile-slab foundations, e.g. in rebuilding the pile-shell ones is an effective engineering solution.

Obviously, the qualitative results obtained on the given models can help determine the patterns of interaction between the existing foundations and soil beds. However, applicability of the proposed method, which proved its effectiveness in new construction, is obvious for buildings under reconstruction and requires further investigation.

5. Acknowledgement
The authors would like to thank the Geotechnics Department, Industrial University of Tyumen, Russia for providing laboratory facilities and giving a helping hand in conducting of this research work.

6. Conflicts of Interest
The authors declare no conflict of interest.

7. References


Compressive Strength and Elastic Modulus of Slurry Infiltrated Fiber Concrete (SIFCON) at High Temperature

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Abstract

SIFCON is a special type of fiber reinforced concrete (FRC) with an unattached fiber matrix that gives the composite matrix important tensile properties and, due to its high fiber content, SIFCON also has distinctive and unique ductility and energy absorption properties. Higher temperature resistance is one of the most important parameters affecting the durability and service life of the material. In this research, the compression strength and elastic modulus of Slurry Infiltrated Fiber Concrete (SIFCON) were tested both before and after exposure to high temperatures. Two fire exposure durations of 2 and 3 hours are examined. In addition to room temperatures, three temperature ranges of 400 °C, 600 °C and 900 °C have been introduced. The results of the experiment showed that the compressive strength and elastic modulus decreased after exposure to high temperatures. The drastically reduction of compressive strength took place with increasing temperature above 600 °C. While, the reduction in elastic modulus values is more significant than the decrease in compressive strength at the same fire flame temperatures. The residual compressive strength and elastic modulus at 900 °C were in the range of (52.1% to 59.6%) and (30.6% to 34.1%) respectively.

Keywords: SIFCON; Steel Fiber; Fire; Elastic Modulus; Compressive Strength.

1. Introduction

Slurry infiltrated fiber concrete (SIFCON) was initially developed in 1979 by Lankard Materials Laboratory, Columbus, Ohio, USA, by incorporating large quantities of steel fibers into reinforced cement composites [1]. SIFCON can be considered as a special type of fiber reinforced concrete (FRC) with a high fiber content where the fibers are molded and infiltrated with cement-based slurry or flowing mortar. Despite the high cost of SIFCON, it is more widely used throughout the world, especially in explosive and impact structures. This is because most of the mechanical and durability properties of these materials are better than those of conventional FRC [2]. SIFCON has a very good application potential in an areas where impact resistance and high ductility are required, especially in the design of the seismic retrofit, in the structures under impact and explosive effects and in the repair of the reinforced concrete structural element. In general, the conventional fiber reinforced concrete (FRC) contains fibers (1–3) % by volume, whereas (SIFCON) contains (4–20) % of fibers. Even though the current practical ranges from (4 to 12) % [3]. SIFCON matrix is a cement slurry or flowing mortar different from the aggregate concrete used in FRC. Thus, SIFCON’s production varies from FRC, which is produced by adding fibers to the fresh concrete, while SIFCON is produced by infiltrating a bed of pre-placed fibers with cement slurry and tightly packed in the mold [2, 4].

The mechanical properties of SIFCON members are evaluated by Shanthini and Mohanraj [4]. The cement-based slurry used was a mixture of different percentages of cement, fly ash, silica fume and eco sand to soil. In this study, the

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water cement ratio of 0.5 and super plasticizer was adopted 2% by weight of cement. The 8% fiber content by volume implemented for SIFCON, distributed randomly. The compressive and splitting tensile strength test results after 28 days of curing were (68.5, 65, 66.5) MPa and (9.86, 9.2, 8.87) MPa, respectively, for the three slurry mix ratio. The researchers found that the members of SIFCON are much better than conventional concrete.

Salih, Frayeh et al. [5] study the effect of hooked ended steel fiber content and mineral admixture replacement silica fume (SF) on strength and deformation characteristics of SIFCON specimens under flexural loading. In this investigation, three volume fractions of steel fiber (6, 8.5, and 11) percent were used. The amount of SF replacement in SIFCON slurry was (10 percent) by weight of cement. Monitoring samples of 100\*100\*400 mm at the age of 7 and 28 days carried out both the flexural strength and toughness characteristic. The findings of these experiments were compared as control samples with those conducted on standard fiber-reinforced mortar (FRM) with 2 percent fiber content. The test results indicate superior properties of SIFCON, as compared to normal FRM, which was positively affected by the use of cement materials (SF) as a partial substitute by cement weight and by increasing the steel fiber volume fraction. The value of flexural strength and toughness up to (28.08 MPa) and (159 N.mm) was obtained at 28 days of age respectively.

The inclusion of steel fibers in concrete has been recognized as a means of improving cracking resistance. Over the past three decades, steel fiber has increasingly been used to enhance refractory concrete performance in many applications as it extends service life. Fiber was widely used to improve concrete ductility. The ability of steel fibers has been shown particularly, to enhance the resistance of spalling of refractory concrete [6]. It is therefore recognized that steel fibers could help enhance the fire efficiency of SIFCON components by delaying concrete spalling. Thus indirectly helping to confinement concrete under compression load [7]. Therefore, it is proposed to conduct an experimental investigation to study the behavior of SIFCON samples at high temperatures.

Khaliq and Kodur [8], examine the effect of temperature on self-consolidating concrete (SCC) and fiber-reinforced SCC (FRSCC) thermal and mechanical properties. Specific heat, thermal conductivity and thermal expansion were measured for thermal properties, whereas compressive strength, tensile strength and elastic modulus were measured for mechanical properties in the 20–800 °C temperature range. The test program considered four SCC mixes, SCCplain, steel, polypropylene, and SCC reinforced hybrid fiber. Results from mechanical property tests show that steel fibers improve the splitting tensile strength at high temperature and the SCC elastic module FRSCC thermal expansion is also slightly higher than SCC thermal expansion at a range of 20–1000 °C.

Beglarigale, Yalçınkaya et al. [9] studied SIFCON's high-temperature composite flexural performance. The scope of this research was subjected to normal or steam cured slurry infiltrated fiber concrete (SIFCON) and slurry samples at 300, 600, 750 and 900 °C. Exposing the samples to 300 °C improved mechanical performance whereas higher temperatures have detrimental effects on SIFCON composites such as loss in the steel fibers cross section and degradation of C–S–H structure.

Exposure to high temperatures, mainly caused by accidental fires, is one of the most severe conditions for building and structural damage. The fire resistance and post-heat exposure behavior of the structural component depends on the thermal and mechanical properties of the composite materials. Elasticity is one of the important material properties that play an important role in the structural behavior of concrete elements both before and after exposure to elevated temperatures [10]. For reasons such as fire or explosion, SIFCON used to strengthen work and military systems may be exposed to high temperatures. High temperature exposure is one of the most important physical degradation processes affecting the life of cement-based composites [9]. Since SIFCON is a relatively new building material, little information on the characteristics of SIFCON is known from previous studies. From this point of view, the primary objective of this research is to perform an experimental investigation to study the effect of fire exposure on the compressive strength and elastic modulus of SIFCON samples after high temperature exposure. The authors believe that this detail study dealing with the performance of slurry infiltrated fiber concrete (SIFCON) samples exposed to direct fire flame is carried out for the first time and will be very useful to concrete technology.

2. Materials and Mix Proportions

A variety of trail slurry mixtures have been used to find an appropriate mixture with optimal characteristics in terms of viscosity, fluidity and filling ability without bleeding or segregation or honeycombing in the fiber network, which causes a dramatic reduction in the mechanical properties of SIFCON. The properties of the mixture are shown in Table 1. Workability of SIFCON slurry is determined by the mini-sluice flow test with accordance to ASTM C1437-15 as 260 mm. Viscosity of the slurry is also assess by Mini V-funnel test and the time measured was 11 s. The overall experimental investigation is shown in the flowchart given in Figure 1.

The physical, chemical and mechanical properties of ordinary Portland cement (OPC) Type I which complies with the ASTM C150-18 specification, used in this investigation are listed in Table 2. The silica fume (SF) utilized in this investigation conforms to the ASTM C1240-15 requirements, and the chemical compositions of it are also given in
Table 2. Fine sand with a maximum size of 1.18 mm and specific gravity of 2.60 is used as fine aggregate for SIFCON. It must be small enough to achieve full infiltration without clogging through the dense steel fiber. Figure 2 shows the grading curve of the natural sand.

![Flow chart of the research plan](image)

**Figure 1. Flow chart of the research plan**

**Table 1. Mix design for SIFCON matrix**

<table>
<thead>
<tr>
<th>Composition Type</th>
<th>Mix Proportion</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>872.4 (kg/m³)</td>
</tr>
<tr>
<td>SF</td>
<td>96.9 (kg/m³)</td>
</tr>
<tr>
<td>Sand</td>
<td>969 (kg/m³)</td>
</tr>
<tr>
<td>Steel Fibre</td>
<td>468 (kg/m³)</td>
</tr>
<tr>
<td>(SP) by the Weight of Cementitious</td>
<td>3.7 (%)</td>
</tr>
<tr>
<td>Water/binding</td>
<td>0.33</td>
</tr>
<tr>
<td>Mini-slump flow</td>
<td>260 (mm)</td>
</tr>
<tr>
<td>V-funnel time</td>
<td>11 (s)</td>
</tr>
</tbody>
</table>
In order to achieve the desired slurry workability, a polycarboxylate super plasticizer (SP) type which conforms to ASTM C494/C494M-17 was added to the mixture to ensure the passing of it through the fiber network without leave honeycombs, where the SP makes it sufficiently liquefied. In addition, two types of steel fiber, which differ in shape and aspect ratio, were used in this research. The first type was hooked end steel fibers with (0.5 mm) diameter, (30 mm) length, and a 60 aspect ratio with a tensile strength greater than 1100 MPa. The second type was straight steel fiber (micro steel fiber) having length of (13 mm), diameter (0.2 mm), and above 2,500 MPa tensile strength with 65 aspect ratio. Volume fraction of steel fiber ($V_f$) used in this study was 6% (3% hooked end steel fiber and 3% micro steel fiber). Fibre volume was calculated according to the volume of the mold for each specimen. Figure 3 illustrate the micro steel fibers and hooked end steel fiber used in this study.

The compression strength ($f_{cu}$) test was performed using ($100\times100\times100$) mm cubes in compliance with ASTM C109/C 109M. While the static modulus of elasticity was carried out in accordance with ASTM C469-87 and using cylinders with a length of 200 mm and a diameter of 100 mm. The mortar must be sufficiently flowable to ensure penetration through the fiber as shown in Figure 4. After casting, samples were demolded after keeping it at $20 \pm 2 \degree C$ for 24 h in saturated humid air. The samples have been cured in water for 28 days with $20 \pm 2 \degree C$ water curing temperature.

### Table 2. Chemical, Physical and mechanical properties for cement and silica fume

<table>
<thead>
<tr>
<th>Chemical composition (%)</th>
<th>Cement</th>
<th>Silica Fume (SF)</th>
<th>Physical properties</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO2</td>
<td>65.0</td>
<td>89.41</td>
<td></td>
</tr>
<tr>
<td>Al2O3</td>
<td>20.6</td>
<td>0.63</td>
<td>SF</td>
</tr>
<tr>
<td>Fe2O3</td>
<td>3.7</td>
<td>0.45</td>
<td>Specific gravity</td>
</tr>
<tr>
<td>CaO</td>
<td>3.5</td>
<td>0.82</td>
<td>Specific surface (m²/kg)</td>
</tr>
<tr>
<td>SO3</td>
<td>2.5</td>
<td>0.87</td>
<td></td>
</tr>
<tr>
<td>L.O.I</td>
<td>3.5</td>
<td>4.10</td>
<td>SF</td>
</tr>
<tr>
<td>CaO (free)</td>
<td>1.32</td>
<td>2.15</td>
<td>Specific gravity</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Specific surface (m²/kg)</td>
</tr>
</tbody>
</table>

![Figure 2. The grading curve of the original sand](image)
The samples were heated to three temperature levels of (400, 600 and 900 °C) for 2 and 3 hours. For compressive strength, three cubes were tested, and three cylinders for elastic modulus at each temperature level and fire exposure duration from each series. Three samples at room temperature were used as reference samples for each test of each series. Thus, 24 cubes and 24 cylinders of each series were tested.

The fire exposure tests were performed at Babylon University Faculty of Engineering in the laboratory of structural material properties. The dimensions of the manufactured furnace is 1500 mm length, 1500 mm width and 1250 mm height. The thickness of all walls for every side is fixed at 250 mm, the principal structure is built with refractory bricks and mortar and has a small openings for giving suitable burners air, and the cover of the furnace has been made with 8 mm thick insulating plate to maintain the fire temperature. The primary aim of the furnace chamber is to raise the levels of temperature of the exposure to fire and to keep it constant for the required duration. To control the temperature of fire, the process of heating comprises of the equipment as illustrated in Figure 5. The burners set comprises of group liquefied petroleum gas burners, all burners were connected together in one pipeline as shown in Figure 5.
In order to control the gas of liquefied petroleum, the electrical gas regulator together with the thermocouples were attached to a digital gauge. The SIFCON samples (for compressive strength and modulus of elasticity) were burned in the furnace according to the standard fire curve recommended of ISO-834 as shown in Figure 6. The first part of fire curve was observed to be similar to that of standard fire curve up to 900 °C. After burning to the specific fire temperature (see part I in Figure 6), columns were held at these maximum temperature for 2 or 3 hours (see part II) at (400, 600 and 900 °C). Under laboratory conditions, column specimens were then cooled down. The heating rate of the experimental curve and the maximum temperature fire exposure is less than that of the ISO-834 recommendation, which is a limitation of the equipment available.

![Figure 6. Experimental and ISO-834 Standard recommended temperature-time curves](image)

The real time-temperature curve was programmed by MATLAB program so that the heat thermocouple inside the furnace was connected to the computer to record the temperature values directly with the time. Then at the end of the burning process the temperatures and times data are stored with the final shape of the time-temperature curve on the computer. Figure 7 illustrates an example of the time-temperature curve drawing by MATLAB program.

![Figure 7. An example for record and drawing of the time-temperature curve by MATLAB program](image)
3. Results and Discussion

In this research, compressive strength and modulus of elasticity of slurry infiltrated fibrous concrete (SIFCON) were investigated both at ordinary temperatures and after exposure to high temperatures.

3.1. SIFCON Compressive Strength

Table 3 demonstrates the SIFCON compressive strength before and after burning. In this table, each value reflects the average values collected from the 3 cubes test in order to minimize the anticipated mistake in any measured consequence.

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>Compressive Strength $f_{cu}$ (MPa)</th>
<th>2 hr. fire exposure</th>
<th>3 hr. fire exposure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>25 °C</td>
<td>400 °C</td>
<td>600 °C</td>
</tr>
<tr>
<td>28</td>
<td>98.41</td>
<td>90.20</td>
<td>79.38</td>
</tr>
<tr>
<td>$f_{cu}/f_{cu}(T=25^\circ C)$ %</td>
<td>100</td>
<td>91.6</td>
<td>80.6</td>
</tr>
<tr>
<td>Change in $f_{cu}$ %</td>
<td>0</td>
<td>-8.4</td>
<td>-19.4</td>
</tr>
</tbody>
</table>

Figure 8 reveals the measured relative residual compressive strengths of SIFCON samples as a function of maximum temperature up to 900 °C. From this Figure, it can be concluded that the drastically reduction of compressive strength took place with increasing temperature above 600 °C. This reduction could be explained by the decomposition of C-S-H and CH hydrates throughout the range (400-600 °C) resulting in a strength decreasing. Further decline was due to the aggregate phase transformation at 573 °C from α to β phase [11]. In addition, the bond between slurry and steel fibers has worsened deteriorated over 700 °C, owing to different expansion among them. Severe cracking at 900 °C led in more than (45%) loss of compressive strength.

Figure 9 shows the comparison between the residual in compressive strength after exposure to fire flame obtained in this study with the residual in compressive strength of different concrete obtained from different studies as follow: (The results of three hours fire exposure at age of 28 days were compared).

- Fitting model without fiber content for normal strength concrete (NSC) and high strength concrete (HSC) [12].
- High-performance concrete (HPC) fitting model [8].
• Steel fiber-reinforced normal strength concrete (SFRC) fitting model [13].
• Experimental results of reactive powder concrete (RPC) [14].

![Figure 9. Comparison of compressive strength values at 28 days with other types of concrete for different researchers](image)

From Figure 9, it is evident that SIFCON performs better than other traditional types of concrete due to its superior microstructure and effective role of steel fibres to resist spalling and crack propagation at high temperatures, resulted in improved compressive strength. The results of this research are in good agreement with the SFRC results as reported by Aslani and Samali, (2014) [13].

The appearance of the cube samples after fire flame exposure at various temperature levels and fire durations are demonstrated in Figure 10, the pictures of the fire exposed samples displaying distinct micro cracks appearances without the appearance of spalling at different temperature levels taken by HD digital camera.

![Figure 10. The appearance of cube samples after fire flame exposure for 3 hr. fire duration at various fire temperature levels](image)
3.2. Static Modulus of Elasticity

The modulus of elasticity is strongly influenced by the concrete materials and their proportions. An increase in the modulus of elasticity is expected with an increase in compressive strength since the slope of the ascending branch of the stress-strain diagram becomes steeper.

Figure 11 depicts the relationship between modulus of elasticity and fire flame temperature levels for SIFCON samples. From the results of Table 4, it can be observed that the reduction in elastic modulus values at the same fire flame temperatures is more significant than the reduction in compressive strength. Reducing the elastic modulus is due to the breakage of bonds in the cement paste microstructure and the differential movement between the cement paste and the aggregate that occurs when the SIFCON is subjected to high firing temperatures [6].

Table 4. Test values of elastic modulus of SIFCON samples before and after exposure to fire flame

<table>
<thead>
<tr>
<th>Age (days)</th>
<th>25 °C</th>
<th>2 hr. fire exposure</th>
<th>3 hr. fire exposure</th>
</tr>
</thead>
<tbody>
<tr>
<td>28</td>
<td>28.77</td>
<td>20.59</td>
<td>16.37</td>
</tr>
<tr>
<td>Ec/Ec (T=25°C) %</td>
<td>100</td>
<td>71.6</td>
<td>56.9</td>
</tr>
<tr>
<td>Change in Ec %</td>
<td>0</td>
<td>-28.4</td>
<td>-43.1</td>
</tr>
</tbody>
</table>

![Figure 11. Fire flame effect of on elastic modulus of SIFCON samples](image)

In Figure 12, the ratio of modulus of elasticity at elevated temperature to corresponding unheated modulus of elasticity has been plotted along with those computed from experimental values of other type of concrete obtained from relevant literature. (The results of three hours fire exposure at age of 28 days were compared).

- Fitting model for normal strength concrete (NSC) [12].
- Fitting model for plain high performance concrete (HPC) [8].
- Fitting model for steel fiber-reinforced normal strength concrete (SFRC) [13].
- Ultra-high strength concrete (UHSC) experimental results [15].
- Experimental results of reactive powder concrete (RPC) [14].
Figure 12. Comparison of elastic modulus values at 28 days with other types of concrete for different researchers

It is evident from Figure 12 that the elastic modulus of SIFCON samples obtained from the experimental work of the present study is comparable to that of SFRC up to 400 °C, but at higher temperatures, SIFCON performs better due to the interlocking action of the fibres, where crack propagation has been controlled and crack growth has been inhibited. In addition to the effect of steel fibre on the increase in the weight of the specimens, and consequently the increase in the elastic modulus and the effective role of high content of steel fibres in resisting spalling and cracking at high temperatures.

4. Conclusions

The following conclusions can be drawn on the basis of the test results of this study and within the limitations of the test parameters:

- The fire exposed SIFCON samples maintained its shape after burning and no evidence of spalling was found.
- The drastically reduction of compressive strength and elastic modulus took place with increasing temperature above 600 °C due to the decomposition of C-S-H and CH hydrates.
- Results show that the decrease in elastic modulus values at the same fire flame temperatures is more significant than in compressive strength.
- The residual compressive strength and elastic modulus at 900 °C was in the range of (52.1% to 59.6%) and (30.6% to 34.1%) respectively.
- By comparison the results of SIFCON residual compressive strength and elastic modulus after fire flame exposure with the results calculated from experimental values of other type of concrete collected from many other studies. It’s clear that SIFCON performs better than other types of concrete at high temperatures.

5. Acknowledgement

The authors wish to express their gratitude and sincere appreciation to the head and staff of Civil Engineering Department in Babylon University. I would like also to express my special appreciation to staff of construction materials laboratory at the civil engineering department for presenting all the facilities to finish this work.

6. Conflicts of Interest

The authors declare no conflict of interest.
7. References


Weather Impact on Passenger Flow of Rail Transit Lines

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Abstract

Passenger flow prediction is important for the planning, design and decision-making of urban rail transit lines. Weather is an important factor that affects the passenger flow of rail transit line by changing the travel mode choice of urban residents. A number of previous researches focused on analyzing the effects of weather (e.g. rain, snow, and temperature) on public transport ridership, but the effects on rail transit line yet remain largely unexplored. This study aims to explore the influence of weather on ridership of urban rail transit lines, taking Chengdu rail transit line 1 and line 2 as examples. Linear regression method was used to develop models for estimating the daily passenger flow of different rail transit lines under different weather conditions. The results show that for Chengdu rail transit line 1, the daily ridership rate of rail transit increases with increasing temperature. While, for Chengdu rail transit line 2, the daily ridership rate of rail transit decreases with increasing wind power. The research findings can provide effective strategies to rail transit operators to deal with the fluctuation in daily passenger flow.

Keywords: Weather Effect; Rail Transit Line; Passenger Flow; Estimation Model.

1. Introduction

The passenger flow estimation of urban rail transit is widely used as the foundation for the planning, design, and daily operations of rail transit. Weather can influence people’s travel behavior and traffic safety, and then have an impact on passenger flow of rail transit line. For example, rain is considered one of the most common adverse weather that may lead to change or cancellation of trips. But, weather factors are not usually presented in the existing models for estimating rail transit line ridership, which results in an insufficient or excessive estimation in the design stage, and unexpected large fluctuations in operation stage. It is essential to identify the impacts of weather factors on passenger flow of rail transit line. The relevant research mainly includes three aspects: data preprocessing of passenger flow [1, 2] quantitative analysis of impact factors [3-7] and development of estimation models [8, 9].

Several studies have explored the effects of rain and snow on public transit ridership. Inclement weather has an impact on people’s travel modes and travel routes, and further effects on passenger flow in public transport [10, 11]. Changnon [12] found that summer rain days have a reduced number of passengers using public buses compared to summer sunny days. Cravo et al. [13] found that rain and snow have negative impacts on passenger flow of bus and subway. Guo et al.
investigated the impact of weather elements, and revealed that rain has a negative impact on bus and rail ridership. Zhou et al. [15] found that the negative impact of rain on bus ridership appears obvious during off-peak time, and no significant effect shows during peak hours.

Arana et al. [16] showed increasing temperature leads to an increase in bus ridership on weekends. However, other studies show inconsistent results regarding the impact of temperature on passenger flow. For example, Kashfi et al. [17] exhibited no obvious relationship between temperature and bus ridership. Stover et al. [18] found that snow temperature has no obvious impact on bus passengers.

Some studies focused on developing the estimation models for daily passenger flow. Cravo et al. [13] used a cross-sectional regression model to determine the impact of weather on New York City Transit’s daily ridership. Stover et al. [18] used the least square methods to analyze the impact of weather on bus ridership. Zhao et al. [19] identified the impact of weather factors on passenger flow rate using multiple linear regression.

Most existing research has focused on the effects of weather (e.g. rain, snow, and temperature) on public transport ridership, but the effects on rail transit remain largely unexplored. On the other hand, little evidence is available to examine the weather-transit ridership in different lines of urban rail transit. Hence, there is a need for transport scholars to begin to determine the effects of weather on rail transit ridership in multiple lines. This study will identify the impacts of weather on ridership of rail transit line, in the two rail transit lines in Chengdu. Six months of data will be used to model the relationship between weather and ridership with linear regression method.

2. Research Methodology

2.1. Data Collection

2.1.1. Passenger Flow Data and Weather Data

The datasets used in this study consist of two types: passenger flow data and weather data. The passenger flow data covering a period from Jan 01, 2017 to Jun 01, 2017 was obtained from China Railway Corporation official website. It should be noted that Chengdu opened a new rail transit line on Jun 02, 2017, which led to a significant change of passenger flow. Therefore, the 2017 data after Jun 02 was excluded. The average daily passenger flow was calculated by weekday, weekend and holiday, shown in Table 1. The data were collected from two Chengdu rail transit lines, lines 1 and 2 (see Figure 1). It was observed that the daily average passenger flow is higher on weekday than on weekend, and weekend is higher than holiday in the two lines.

![Figure 1. Chengdu rail transit lines 1 and 2](image-url)
Table 1. Passenger flow of rail transit in the two lines

<table>
<thead>
<tr>
<th>Line</th>
<th>Average daily passenger flow (thousand)</th>
<th>Weekdays</th>
<th>Weekends</th>
<th>Holidays a</th>
</tr>
</thead>
<tbody>
<tr>
<td>Line 1</td>
<td>722</td>
<td>517</td>
<td>391</td>
<td></td>
</tr>
<tr>
<td>Line 2</td>
<td>663</td>
<td>628</td>
<td>543</td>
<td></td>
</tr>
</tbody>
</table>

Note: a-Include seven national holidays in China, see Table 2 for details.

The meteorological data from Jan 01, 2017 to Jun 01, 2017 were gained from the World Weather Online [20], including daily temperatures (°C) (the lowest and highest), weather conditions (sunny, cloudy, rainy and snowy), wind speed (m/s), cloud fraction (%), precipitation (mm), air pressure (bar) and humidity (%). The flowchart of this research method is available in Figure 2.

![Flowchart of the research method](image)

2.2. Data Processing

2.2.1. Passenger Flow of Rail Transit Line

Before analyzing the relationship between passenger flow and weather, it is necessary to clean raw data. Daily passenger flow of rail transit line is affected by many factors such as holidays, large-scale events, and emergencies. The purpose of data cleaning is not only to detect and correct incomplete or inaccurate records, but also to reduce or eliminate the effects of other factors except weather.

First, it is important to identify the outliers of passenger flow data. Outliers are observations that lie abnormal distances from other values. Fig. 3 shows the daily passenger flow of Chengdu rail transit line 1 between 01/01/2017 and 01/06/2017. It was observed that several obvious outliers are presented at the end of January and the beginning of February. This time period is exactly in the longest national holiday in China (Lunar New Year). China has seven national holidays (shown in Table 2), in which the passenger flow varies significantly. Therefore, the holiday effect needs to be removed from passenger flow data.

![Graph of daily passenger flow](image)

Figure 3. Daily passenger flow of Chengdu rail transit line 1 in 2017 (Jan 01-Jun 01)
Table 2. Holiday schedule in 2017

<table>
<thead>
<tr>
<th>Holidays in China</th>
<th>Date</th>
<th>Total days</th>
</tr>
</thead>
<tbody>
<tr>
<td>New Year's Day</td>
<td>2016.12.31~2017.01.02</td>
<td>3</td>
</tr>
<tr>
<td>Lunar New Year</td>
<td>2017.01.27~2017.02.02</td>
<td>7</td>
</tr>
<tr>
<td>Qingming Festival</td>
<td>2017.04.02~2017.04.04</td>
<td>3</td>
</tr>
<tr>
<td>May Day</td>
<td>2017.04.29~2017.05.01</td>
<td>3</td>
</tr>
<tr>
<td>Dragon Boat Festival</td>
<td>2017.05.28~2017.05.30</td>
<td>3</td>
</tr>
<tr>
<td>Mid-autumn Festival and National Day</td>
<td>2017.10.01~2017.10.08</td>
<td>8</td>
</tr>
</tbody>
</table>

Table 3. Standard deviations of data sequences removing days before or after holiday

<table>
<thead>
<tr>
<th>Line</th>
<th>Days eliminated</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SD</td>
<td>SD</td>
<td>SD</td>
<td>SD</td>
<td>SD</td>
<td></td>
</tr>
<tr>
<td>Line 1</td>
<td>After holiday</td>
<td>14.82</td>
<td>14.96</td>
<td>15.13</td>
<td>15.19</td>
<td>15.33</td>
</tr>
<tr>
<td>Before and after holiday</td>
<td>14.82</td>
<td>14.74</td>
<td>14.98</td>
<td>15.21</td>
<td>15.55</td>
<td></td>
</tr>
<tr>
<td>Before holiday</td>
<td>9.38</td>
<td>8.85</td>
<td>8.85</td>
<td>8.96</td>
<td>9.08</td>
<td></td>
</tr>
<tr>
<td>Before and after holiday</td>
<td>9.38</td>
<td>8.92</td>
<td>9.03</td>
<td>9.20</td>
<td>9.47</td>
<td></td>
</tr>
</tbody>
</table>

Note: SD= Standard Deviation

It was also noticed that significant changes in passenger flow usually present in several days before or after a holiday. To determine the days, the standard deviations of data sequences were compared. Standard deviation is the measure of dispersion of a set of data from its mean. The smaller the standard deviation, the lower the dispersion. Results show that the standard deviation of the dataset decreases, excluding the first day before holiday, and the first day before and after holiday, compared to the original dataset. When two or more days before and after holiday are removed, the standard deviations do not change much. For example, the standard deviation of line 1 reduces from 14.82 to 14.61 after removing the first day before holiday. And then, the standard deviation begins to slightly increase after elimination of two or more days before holiday. Therefore, to keep the original data as much as possible, only data of the first day before holiday were selected to eliminate (shown in Table 3).

2.2.2. Weather Data

In this study, weather data was cleaned in the following three aspects:

1. Eliminate the data whose corresponding passenger flow data were removed.
2. Eliminate the data with missing values.
3. Eliminate the data of extreme weather events, such as hailstorm.

2.3. The Estimation Model for Daily Passenger Flow

2.3.1. Correlation Analysis between Weather and Passenger Flow of Rail Transit Lines

Because the passenger flow distribution presents a significant difference on weekend and on weekday, the flow data of weekend and weekday were separated. Scatterplots were used to analyze if there are relationships between weather factors and passenger flow of rail transit line. In Figure 4, it was found that temperature (the average of the highest and lowest temperature) has an effect on passenger flow of rail transit line 1, as well as wind speed has an influence on passenger flow of rail transit line 2. It was also observed that the weekend flow is more likely to be affected by weather than the weekday flow.

2.3.2. Models of Daily Passenger Flow Estimation

The daily passenger flow presents a periodic fluctuation between weekday and weekend, and obvious changes in holidays (shown in Figure 3). Thus, as developing the daily passenger flow estimation model, the day factor (DF, weekday and weekend) and holiday factor (HF) should be included. Otherwise,
Temperature Factor (TF) and Wind Power Factor (WPF); it was converted by wind speed) should be also included due to their impacts on passenger flow.

Based on Singhal’s study [1], a multiple linear regression model was established for estimating daily passenger flow in Chengdu rail transit lines. In the model, the dependent variable is daily passenger flow, and the independent variables include the four influencing factors, as shown in Equation 1.

\[
DR = \alpha + \beta_1 DF_{cb} + \beta_2 HF_{cd} + \beta_3 TF_{ct} + \beta_4 WPF_{cwp}
\]  

(1)

Where DR estimated daily passenger flow of rail transit is line; \(\alpha\) is model constant. \(\beta_1, \beta_2, \beta_3, \beta_4\) is coefficient for estimation. \(DF_{cb}\) is day factor for rail transit line \(c\) ∈ \{line 1, line 2\} for day type \(b\) ∈ \{Mon, Tues, Wed, Thu, Fri, Sat, Sun\}. \(HF_{cd}\) is holiday factor for rail transit line \(c\) during holiday \(d\). \(TF_{ct}\) is temperature factor for rail transit line \(c\) during temperature interval \(t\) ∈ \([0, 10)∪(10, 20)∪[20, 30)\). \(WPF_{cwp}\) is wind power factor for rail transit line \(c\) having a wind power \(wp\) ∈ \{light air and lower, light air to light breeze, gentile to moderate breeze\}. 

Figure 4. Effects of temperature and wind speed on passenger flow of Chengdu rail transit lines

- Temperature Factor (TF) and Wind Power Factor (WPF); it was converted by wind speed.
- Based on Singhal’s study [1], a multiple linear regression model was established for estimating daily passenger flow in Chengdu rail transit lines. In the model, the dependent variable is daily passenger flow, and the independent variables include the four influencing factors, as shown in Equation 1.

\[
DR = \alpha + \beta_1 DF_{cb} + \beta_2 HF_{cd} + \beta_3 TF_{ct} + \beta_4 WPF_{cwp}
\]  

(1)

Where DR estimated daily passenger flow of rail transit is line; \(\alpha\) is model constant. \(\beta_1, \beta_2, \beta_3, \beta_4\) is coefficient for estimation. \(DF_{cb}\) is day factor for rail transit line \(c\) ∈ \{line 1, line 2\} for day type \(b\) ∈ \{Mon, Tues, Wed, Thu, Fri, Sat, Sun\}. \(HF_{cd}\) is holiday factor for rail transit line \(c\) during holiday \(d\). \(TF_{ct}\) is temperature factor for rail transit line \(c\) during temperature interval \(t\) ∈ \([0, 10)∪(10, 20)∪[20, 30)\). \(WPF_{cwp}\) is wind power factor for rail transit line \(c\) having a wind power \(wp\) ∈ \{light air and lower, light air to light breeze, gentile to moderate breeze\}.
Based on Kashfi’s study [21], day factor (DF\textsubscript{cb}) and holiday factor (HF\textsubscript{cd}) for two rail transit lines were calculated by Equations 2 and 3 respectively, and the calculated values are shown in Table 4.

\[
DF_{cb} = \frac{\sum_{i=1}^{N_b} DR_{cb,i}}{N_bDR_{cav}}
\]  

(2)

Where \(DR_{cb,i}\) is original daily passenger flow of rail transit line \(c\) on day \(i\) of day type \(b\). \(N_b\) is the number of relevant days of day type \(b\). \(DR_{cav}\) is original average daily passenger flow of rail transit line \(c\).

\[
HF_{cd} = \frac{\sum_{i=1}^{N_d} DR_{cd,i}}{N_dDR_{cav}}
\]  

(3)

Where \(DR_{cd,i}\) is original daily passenger flow of rail transit line \(c\) on day \(i\) of holiday \(d\). \(N_d\) is the number of relevant days of holiday \(d\).

Temperature factor (TF\textsubscript{ct}) and wind power factor (WPF\textsubscript{ctwp}) for the two rail transit lines were calculated by Equations 4 and 5 respectively, and the calculated values are demonstrated in Table 5.

\[
TF_{ct} = \frac{\sum_{i=1}^{N_t} DR_{ct,i}}{N_tDR_{cav}}
\]  

(4)

Where \(DR_{ct,i}\) is original daily passenger flow of rail transit line \(c\) on day \(i\) for a given temperature interval \(t\). \(N_t\) is the number of relevant days occurring in a temperature interval \(t\).

\[
WPF_{ctwp} = \frac{\sum_{i=1}^{N_{wp}} DR_{cwp,i}}{N_{wp}DR_{cav}}
\]  

(5)

Where \(DR_{cwp,i}\) = original daily passenger flow of rail transit line \(c\) on day \(i\) for a given wind power \(wp\); \(N_{wp}\) is the number of relevant days having a wind power \(wp\).

<table>
<thead>
<tr>
<th>Line</th>
<th>Weekday</th>
<th>Weekend</th>
<th>Holiday</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mon</td>
<td>Tues</td>
<td>Wed</td>
</tr>
<tr>
<td>Line 1</td>
<td>1.15</td>
<td>1.14</td>
<td>1.15</td>
</tr>
<tr>
<td>Line 2</td>
<td>1.03</td>
<td>1.02</td>
<td>1.03</td>
</tr>
</tbody>
</table>

Table 5. Calculated values of temperature factor (TF\textsubscript{ct}) and wind power factor (WPF\textsubscript{ctwp}) for two lines

<table>
<thead>
<tr>
<th>Line</th>
<th>Temperature (°C)</th>
<th>Wind power</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0~10</td>
<td>10~20</td>
</tr>
<tr>
<td>Line 1</td>
<td>1.04</td>
<td>1.06</td>
</tr>
<tr>
<td>Line 2</td>
<td>0.99</td>
<td>1.03</td>
</tr>
</tbody>
</table>

3. Results and Discussion

SPSS software was used to perform multiple linear regression, and stepwise regression method was used to eliminate non-significant variables. Table 6 shows the parameter estimates for the regression model, including the correlation coefficient (B), standard error (S.E), t-statistics, P value and R\(^2\) value. R\(^2\) is a goodness-of-fit measure for the model: the higher the R\(^2\) value, the better the estimation model fits the passenger flow data. If R\(^2\) value is more than 0.8, this value is generally considered strong effect size. If |t| is greater than 1.96 at a 5\% significance level, we are 95\% confident that the variable has a significant impact on the daily passenger flow in rail transit lines, otherwise the variable will be eliminated.

It was seen that the day and holiday variables have high coefficients, which indicates that they play a dominant role in daily passenger flow (P < 0.001). The temperature has a significant impact on passenger flow for rail transit line 1 (P < 0.05), and the wind power has a significant impact on passenger flow for rail transit line 2 (P < 0.05). According to the |t| value, the variables with a |t| value less than 1.96 were eliminated (see Table 6).
Table 6. Fitting results of regression model

<table>
<thead>
<tr>
<th>Line 1</th>
<th>B</th>
<th>S. E</th>
<th>t</th>
<th>P</th>
<th>Line 2</th>
<th>B</th>
<th>S. E</th>
<th>t</th>
<th>P</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Constant</td>
<td>-41.86</td>
<td>6.33</td>
<td>-5.98</td>
<td>0.00</td>
<td>Constant</td>
<td>-17.92</td>
<td>4.90</td>
<td>-4.48</td>
<td>0.00</td>
</tr>
<tr>
<td>DF&lt;sub&gt;c,b&lt;/sub&gt;</td>
<td>47.68</td>
<td>2.83</td>
<td>18.96</td>
<td>0.00</td>
<td>DF&lt;sub&gt;c,b&lt;/sub&gt;</td>
<td>31.15</td>
<td>5.02</td>
<td>6.01</td>
<td>0.00</td>
</tr>
<tr>
<td>HF&lt;sub&gt;c,d&lt;/sub&gt;</td>
<td>54.66</td>
<td>3.70</td>
<td>16.38</td>
<td>0.00</td>
<td>HF&lt;sub&gt;c,d&lt;/sub&gt;</td>
<td>51.55</td>
<td>4.84</td>
<td>10.45</td>
<td>0.00</td>
</tr>
<tr>
<td>TF&lt;sub&gt;c,t&lt;/sub&gt;</td>
<td>2.97</td>
<td>6.74</td>
<td>2.07</td>
<td>0.042</td>
<td>TF&lt;sub&gt;c,t&lt;/sub&gt;</td>
<td>6.03</td>
<td>6.58</td>
<td>0.00</td>
<td>0.324</td>
</tr>
<tr>
<td>WPF&lt;sub&gt;c,wp&lt;/sub&gt;</td>
<td>7.12</td>
<td>10.29</td>
<td>0.36</td>
<td>0.122</td>
<td>WPF&lt;sub&gt;c,wp&lt;/sub&gt;</td>
<td>-1.20</td>
<td>2.87</td>
<td>-1.99</td>
<td>0.047</td>
</tr>
</tbody>
</table>

Note: Significant level α = 0.05

From the results of regression model fitting, $R^2$ values with more than 0.8 indicate that the model produced a high goodness of fit. In order to improve the accuracy of passenger flow estimation, the daily passenger flow estimation models for Chengdu rail transit line 1 and line 2 were established respectively, based on the results in Table 6, as shown in Equations 6 and 7.

Line 1: $DR = \alpha_1 + \beta_{11}DF_{c,b} + \beta_{12}HF_{c,d} + \beta_{13}TF_{c,t}$  \hspace{1cm} (6)

Line 2: $DR = \alpha_2 + \beta_{21}DF_{c,b} + \beta_{22}HF_{c,d} + \beta_{23}WPF_{c,wp}$  \hspace{1cm} (7)

For Chengdu’s rail transit line 1 and line 2, the estimates using the newly-developed models were compared with the original ridership rates. The estimated daily passenger flow, calculated using Eq. (6) for line 1 and Eq. (7) for line 2 respectively, were compared with the original daily passenger flow (shown in Figs. 5 and 6). It was observed that in general, the estimates for the two lines are in close proximity to the actual values. Line 1 primarily provides commuter service to people, including businessmen, white-collar, IT elite, and business travellers. The target passenger travel occurs mainly on weekday, so there are significant differences between weekday and weekend passenger flows. Weekly patterns are repeated within each month block, causing some peaks and troughs from the actual data were not fully captured. However, Line 2 shows subtle differences between weekday and weekend passenger flows. Similar to Line 1, some peaks and troughs from the actual data were not fully captured.

Figure 5. Time series graph of actual daily passenger flow trend & fitting of Model for rail transit line 1
This study analyzed the effects of temperature and wind power on daily passenger flow in rail transit lines. It was found that temperature has a significant impact on passenger flow for Chengdu rail transit line 1, and wind power has a significant impact on passenger flow for Chengdu rail transit line 2.

3.1. Temperature

In the temperature ranging from 20 to 30°C, temperature has a significant impact on daily passenger flow of Chengdu rail transit line 1, and the correlation coefficient is positive. The results indicate that the daily passenger flow of rail transit line 1 increases with an increase of temperature.

3.2. Wind Power

Within the wind power ranging from level 1 to level 2, wind power has a significant impact on daily passenger flow of Chengdu rail transit line 2, and the correlation coefficient is negative. The results indicate that the daily passenger flow of rail transit line 2 reduces as wind power increases.

4. Conclusion

This study performed large-scale data analysis on the data of daily passenger flow and weather elements to explore the impacts of weather factors on usage of rail transit line. The daily ridership estimation models were established under different weather conditions for different rail transit lines. The results show that in Chengdu, the increase in temperature is associated with increasing ridership in rail transit line 1, and the increase in wind power is associated with decreasing ridership in rail transit line 2. These findings provide rail transit operators with valuable information to deal with daily passenger flow fluctuation related to varying weather conditions.

5. Funding

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6. Conflicts of Interest

The authors declare no conflict of interest.

7. References


Assessment of SMC Frames under Different Column Removal Scenarios

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Abstract

Throughout the past decades, failure of structures threatening the lives of humans had been popular whether through structure failure due to human error such as Hyatt Regency walkway collapse, 1981, terrorist attacks on the American embassy attack in Nairobi, Kenya 1998 and the famous 9/11 attacks in 2001 and many more. As a result of these incidents, The Unified Facilities Criteria (UFC) was developed concerning the progressive collapse issues by analyzing different types of structures under column loss and studying the overall structural behavior. However, the (UFC) didn’t scope on the local behavior of the structural components and its connection under column loss. In this research, the main objective is to study the local behavior of the special moment frame connection (SMC) under column loss. A detailed study is conducted on a 3D model fully designed by adopting the strong-column weak-beam approach following the ACI318-14 regulations. Two frames are selected from the designed structure, interior and exterior frames, to apply the column loss scenario in different locations and different floor levels. The Applied Element Method is adopted in the study. Non-linear time-dependent dynamic analysis is implemented to apply the different column removal scenarios. Twelve case studies are modeled in detail using the Extreme Loading for Structures (ELS) software at which all elements are modeled and analyzed in a 3D model technique. After analyzing the different case studies, structure behavior is observed. Some cases encountered total collapse, other cases encountered partial /local collapse and finally, some survived the column loss scenario. Many parameters are involved and studied in the research. Failure pattern is observed for collapsed cases, the cause of failure is monitored and studied. Special moment connection behavior is studied concerning the shear connection capacity. The location of the column removal with the type of frame selected played an important role in changing the structural behavior from one case to another. As a result, it is not applicable to assume that due to the special moment connection ductility, the structure will be able to resist the column loss in all cases.

Keywords: Special Moment Connections; Column Loss; Connection Failure Pattern; Ductile RC Frames; Applied Element Method.

1. Introduction

For the past few years, the progressive collapse of structures took place as a result of losing many lives. This led to many codes and parties to study the Progressive Collapse of the structures and set many regulations either to resist it or to restrict its effect on some parts of the buildings. Progressive collapse can take place due to losing one of the main elements either due to terrorist attack, blasting or structural overloading. Regulations that are set to resist the progressive collapse of structures concentrated on the overall structural behavior and how to bridge the failed element and redistribute the loads on the surrounded elements regardless of the structural system used or the type of connections involved in the study. As a result, it is essential to consider the structure system as a main parameter in affecting the

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structural behavior due to column loss. Ductile Special moment frame connections are one of the primary systems used in resisting earthquake loading due to its ductility and redundancy.

Many experimental research approaches are conducted to study the structural behavior under different column loss scenarios, mostly for steel frames and connections. Few experimental kinds of research are performed for reinforced concrete frame connections due to the limitations that face the experimental testing such as; the specimen size, column removal scenario simulation and finally the accuracy of the results measured from the failed specimen. The numerical analysis took place and validated by experimental testing to find the most suitable way to investigate the structural behavior due to progressive collapse without the experimental limitations.

Guo et al. [1] observed the behavior of the semi-rigid steel connection under a single column removal scenario; experimental examination took place with a pseudo-static test of a composite frame with flush-endplate connections under the loss of middle column is carried out. He compared the experimental testing with a Finite Element Method (FEM) model subject field using both 3D and 2D components. The properties of bolts affected the progressive collapse through the analytical study conducted. They suggested to improve the behavior of connection in resisting progressive collapse. Li and Sasani [2] evaluated the effects of seismic design in reinforced concrete frame structures and its effect in resisting Progressive collapse. The main parameter in the study is the span length and its response under column removal scenario. They focused on shorter span’s buildings at medium seismic activity sites, and taking the effect of the joint torsional stiffness and concrete tensile strength into consideration. The study is implemented using an equivalent single degree of freedom systems to evaluate the maximum displacement response of structures after element failure. It is concluded that the shear resistance of Special frame beams is higher than that of ordinary beams. Weng et al. [3], observed the damage assessment criterion and a conducted an experimental testing on a three-scaled moment-resisting RC frame to validate proposed flexural and axial damage criteria. They concluded from the obtained results that the proposed damage assessment criteria are valid and reliable for the progressive collapse analysis of RC frames. Dinu et al. [4] investigated the performances of four types of beam-to-column steel connections against progressive collapse by constructing Two-span frames and testing it under a central column removal scenario until failure. The tested specimens showed good ductility, with the catenary action making a significant contribution to the ultimate load resistance. The beams’ ultimate rotations were higher than the deformation limit in Unified Facilities Criteria guidelines. Li [5] conducted a push-down analysis experimentally using a 1/3 scale one-story bare steel moment frame substructure. The test results reveal that flexural action plays an essential role in resisting progressive collapse along the entire loading process.

However, the catenary action becomes the primary collapse resisting mechanism in the final stage of loading. Dynamic responses of the test specimen are estimated using the energy-based method. The analysis results suggest that catenary action has a significant impact on the value of the dynamic increase factor under large deformation conditions. The three-dimensional reinforced concrete structure subjected to consecutive column loss scenarios with nonlinear dynamic response. According to Arshian et al. [6], a numerical approach is used to investigate sections subjected to consecutive column loss. It is found that theses sections suffered from more deformation compared to sudden single column failure scenarios.

An Experimental approach was used by Behnam et al. [7] and concluded that the effect of beam width to column width ratio on the seismic behavior of exterior reinforced concrete frame beam-column connection. Four specimens designed under (ACI318-14) and (ACI 352R-02) and tested under reversed cyclic loading conditions; Thus, he concluded that a wide beam could improve the joint shear capacity by enlarging the joint-effective width. Lu et al. [8] studied the interaction between beams and slabs and the two-way load transfer characteristics. Experimental testing was conducted on five 1/3-scaled RC frame substructure specimens, including four beam-slab specimens and one beam specimen without a slab, the slab contribution affected the progressive collapse resistance. The effect of critical structural parameters (i.e., the beam height, slab thickness, and seismic reinforcement) on the collapse resistance was investigated by analyzing the applied loads. A numerical investigation by Pham et al. [9] is conducted using the finite element method to check on the static and dynamic response under progressive collapse. Many verification procedures were performed to check the integrity of the model; in the actual verification, all parameters were investigated and resulted that under blast conditions, catenary action could be changed to prevent the structure collapse even if the bottom longitudinal reinforcement in the beam has already fractured. However, according to the authors, this method may have overestimated the structural resistance if a contact detonation event induces the localized damage. Some researches applied the column loss scenario induced by fire as Li et al. [10] presented it in an experimental study. They applied fire tests on five RC connections with varying reinforcement development lengths, with and without cooling effects using built hybrid heating furnace. They concluded that it is essential to provide practical recommendations for enhancing the structural robustness of structural configurations of the form considered in this study. Some studies scope on the precast connection’s behavior due to progressive collapse, Al-Salloum et al. [11], studied how to strengthen the precast connections using bolted steel plates and Qian et al. [12], studied the resistance of high-performance dry connections due to progressive collapse. Tang et al. [13], studied and designed an asymmetric double-half-span single-column structure with a fully bolted connection in (RCS) frame structure. A static loading test is applied to a penultimate column
to simulate the Colum loss scenario. Experimental and numerical assessment are carried out in the study, and a comparison between the two methods are concluded. It is found that the ultimate rotation angle of the connection obtained experimentally and numerically was higher than the allowable limit in the DoD (Department of Defense) guidelines. And finally concluded that enhancing the steel strength helped to improve the connection resistance towards progressive collapse. Also, Elsanadedy et al. [14], studied and analyzed reinforced concrete (RC) special moment-resisting frame (SMRF) assemblies under column-loss scenarios considering bond-slip effects at the concrete-to-steel rebar interface using non-linear finite element method. They considered different parameters such as; type of assembly, column continuity, and development of beam rebars at exterior joints, applied axial loads on columns and beam continuity at exterior joints and studied its effect on the selected connection due to column loss.

As a result, a research sequence is applied to study the special moment frame connection behavior under column loss scenario as shown in Figure 1.

2. Research Methodology

The Applied Element Method is an innovative modeling method adopting the concept of discrete cracking [15-17]. In Applied Element Method (AEM), the structures are modeled as an assembly of relatively small elements, made by dividing the structure virtually, as shown in Figure 2a. The elements are connected along their surfaces through a set of normal and shear springs. The springs are responsible for the transfer of normal and shear stresses, respectively, from one element to another. Springs represent stresses and deformations of a specific volume as shown in Figure 2b.
Every single element has 6 degrees of freedom; 3 for translations and 3 for rotations. Relative translational or rotational motion between two neighboring elements causes stresses in the springs located at their common face as shown in Figure 3. These connecting springs represent stresses, strains, and connectivity between elements. Two neighboring elements can be separated once the springs connecting them are ruptured.

Fully nonlinear path-dependent constitutive models for reinforced concrete are adopted in the AEM as shown in Figure 4. For concrete in compression, an elasto-plastic and fracture model is adopted, Maekawa [19]. When concrete is subjected to tension, a linear stress-strain relationship is adopted until the cracking of the concrete springs, where the stresses then drop to zero. The residual stresses are then redistributed in the next loading step by applying the redistributed force values in the reverse direction. For concrete springs, the relationship between shear stress and shear strain is assumed to remain linear until the cracking of concrete. Then, the shear stresses drop down as shown in Figure 3. The level of the drop of shear stresses depends on the aggregate interlock and friction at the crack surface. For reinforcement springs, the tangent stiffness of reinforcement is calculated based on the strain from the reinforcement spring, loading status (either loading or unloading) and the previous history of steel spring which controls the Bauschinger’s effect. The solution for the dynamic problem adopts implicit step-by-step integration (Newmark-beta) method Bathe and Chopra [20, 21]. Separated elements may collide with other elements. In that case, new springs are generated at the contact points of the collided elements.
3. Program Validation

In this section, the ELS software is validated by analyzing the specimens tested by Weng et al. [3]. Three one-third scaled specimens are modeled and detailed using the ELS. A mesh sensitivity is conducted to choose the most suitable mesh in simulating the chosen specimen. The tested specimens are identified as Full Restrained (FR), full restrain-seismic (FR-S) and Partially Restrained (PR). The detailing of the tested specimens, as well as the ELS models, are shown in Figures 5 and 6.

3.1. Analysis of Results

Central deflection versus the applied load is shown in Figures 7 to 9. The analytical model shows acceptable results compared to the experimental results. The specimen encountered the same failure pattern as well as the same crack locations as shown in Figure 10.
Figure 9. Load Versus Displacement for PR

Figure 10. Failure in specimens
4. Numerical Case Study

4.1. Characteristics of Structure

A prototype Special moment frame system is adopted in this study as shown in Figures 11 and 12. The structure is two-bay 10 meters length in both directions. The structure is designed to resist lateral loading by considering the weak-beam strong-column approach according to the ACI318-14 [22]. The structure is five stories with a clear floor height of 5 m with a Live load of 2 KN/m². The material properties are taken as follows; 30 MPa for concrete compressive strength and 360 MPa yield strength for all steel bars. Girder and column dimensions with the selected reinforcement are illustrated as shown in Table 1. All slab and beam loads are calculated and applied to the frame girder itself.

<table>
<thead>
<tr>
<th>Exterior Frame</th>
<th>Cross section Dimensions (mm)</th>
<th>longitudinal Reinforcement</th>
<th>Transverse reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>400X1800</td>
<td>4ϕ22 Top &amp; Bottom</td>
<td>5 ϕ 10/m²</td>
</tr>
<tr>
<td>Column</td>
<td>400X1800</td>
<td>12 ϕ 20 Uniformly Distributed</td>
<td>5 ϕ 8/m²</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Interior Frame</th>
<th>Cross section Dimensions (mm)</th>
<th>longitudinal Reinforcement</th>
<th>Transverse reinforcement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Girder</td>
<td>400X1800</td>
<td>4ϕ22 Top &amp; Bottom</td>
<td>5 ϕ 10/m²</td>
</tr>
<tr>
<td>Column</td>
<td>400X1800</td>
<td>12 ϕ 25 Uniformly Distributed</td>
<td>5 ϕ 12/m²</td>
</tr>
</tbody>
</table>
4.2. Numerical Modelling

The structure is modeled in the ELS, two frames; exterior and interior frames, are chosen from the structure and studied separately with different column loss scenarios as shown in Figures 13 and 14. The column loss is applied on different floor levels by applying a load combination as the UFC 4-023-03 regulated [23]. The structure analysis is implemented using two stages of loading, the first stage where all the gravity loads are calculated and applied and the second stage where the column loss simulation took place. The column loss is simulated by removing all the column elements with respect to time starting from stage two in the analysis procedure. All boundary conditions for the columns are set to be fixed to the foundation.

The column loss scenario is applied in a certain pattern. For the exterior and interior frame, the column scenario is involved in two different locations one at a time, middle column loss and edge column loss as shown in different floor levels as shown in Figure 15.

5. Numerical Analysis and Results

The results are presented for four main cases, middle column removal for exterior and interior frames for different floors and corner column removal for the same mentioned frames. Different behaviors are obtained for each case. The failure took place in the case of the Interior frame in most cases; however, no failure took place in the case of the Exterior frame.

The results obtained are in terms of beam deflection, shear applied in the Special moment connections, axial forces in beams, axial forces in columns, normal stresses in the top and bottom reinforcement in the concerned connection along with the redistribution of straining actions due to column loss.
5.1. The Results

5.1.1. Middle Column Removal

**Failure pattern**

The Exterior frame did not fail on the contrary to the Interior frame. This expectancy comes from the fact that the Exterior frame carries half the loads compared to the Interior frame. Figure 16 shows the strain contours and the cause of failure for different cases.

**Beam Deflection**

The displacement is recorded at the location of the column removal. For the exterior frame, deflections vary from 22 mm for the first floor, 34 mm for the third floor and 28 mm for the fifth floor as shown in Figure 17. The first floor encountered the least deflection compared to the other floor levels. For the Interior frame, the first and the third floor failed to resist the column removal; however, the fifth floor stood still and stabilized after reaching 76 mm maximum displacement.
Connection Shear force

The connection shear force after column removal is obtained for both cases. No significant force is applied in the connection of the removed column. For the connections at the sides, the resistance of the structure towards column removal applied additional shear force on the connection as shown in Figure 18.

For the Exterior Frame, the maximum applied shear force in the connection reached 69 tons for the first-floor column removal, 68 tons for the third floor and 71 tons for the last floor column removal consequently. For the interior Frame, a maximum shear of 89.23 ton took place. For the Interior frames, shear forces are obtained at maximum values before the frame loses its stability. The maximum shear applied took place on the third floor with a value of 121.3 tons.

The maximum shear force took place in the column removal floors; this is because the compression arching effect of the beam along with the redistributed bending moment applied additional shear force on the designed connection. The joint capacity calculated previously for design purposes is 460 tons. So, the joint could withstand the applied shear force from column loss for both cases of interior and exterior frames.

![Shear Force in Connection](Figure 18)
Beam Axial Force

The high compression force took place in the girders at the removed column location due to the compression arching effect. In the Exterior frame, the girder reached maximum compression of 450 KN and then sustained the column loss for the first floor, 290 KN for the third floor and 510 KN for the fifth floor.

In the Interior frame, the girder subjected to high compressive stresses of 1500 KN and then due to the high shear force applied, concrete crushing took place and affected the Concrete strength in compression as shown in Figure 19. For the third floor, maximum compression of 600 KN took place before beam failure, and for the last floor level, the beam sustained the column loss and encountered a compression force of a maximum 630 KN and remained stable.

Many parameters affect the frame behavior either to sustain column loss or to collapse. For the Exterior frame, the light loading along with the Vierendeel action helped in resisting and reducing the compressive force applied to the girders. The girders worked as a lateral restraint in cases of the first and third floors and sustained high tensile force shown in Figure 20. In the fifth-floor case, the girder subjected to a higher compression force due to the absence of the Vierendeel effect, as a result of increasing the load on the girder after column loss.

For the Interior frame, although failure took place in cases of first and third column removal, the Vierendeel action contributed to delay the failure for the first floor and increased the applied compression force resulting from the compression arching effect.
Normal Stresses in reinforcement

For the Exterior and interior frames, reinforcement locations studied are taken at the middle connections and the side connection. For each side, top and bottom reinforcement are rerecorded.

- Top Reinforcement

In the Exterior frame, for the three cases, the top reinforcement at the side connection reached its tensile yield stresses of 360 MPa and then stabilized as shown in Figure 21. For the reinforcement at the column removal, the bars encountered compressive stresses due to the change in the structural system from being continuous to a simple beam. Each case faced different compressive stresses as shown in Figure 22. The maximum compressive stresses of 100, 140, and 80 MPa for first, third and fifth-floor column removal took place.

In the Interior frame, at the side connection, for the first floor and the third floor, the reinforcement resisted high tensile stresses till ultimate, then shear failure took place before the rupturing of reinforcement at time 0.66 seconds. For the fifth floor, tensile stresses reached 400 MPa and then stabilized.

At the column removal connection, the reinforcement for the first and third floors encountered maximum compressive stresses of 420 and 360 MPa before the beam shear failure. For the fifth floor, the top reinforcement reached 200 MPa maximum compressive stresses and then stabilized as shown in Figures 21 and 22 respectively.

![Figure 20. Axial force Distribution in Frame girders](image1)

![Figure 21. Normal Stresses in Top Side RFT versus Time at sides](image2)
The normal stresses in bottom reinforcement for both Exterior and Interior frames are calculated as shown in Figures 23 and 24. For the bottom reinforcement at the sides, the reinforcement encountered compressive stresses due to the applied negative bending moment on the connection. For the Exterior frame, the maximum stresses of 200 MPa took place for the third floor and then stabilized.

For the Interior frame, the bottom reinforcement failed for the first and third cases. The first floor failed earlier after reaching maximum stresses of 498 MPa. The fifth floor reached 360 MPa and then stabilized.

At the column removal location, the bottom reinforcement of the Exterior frame reached maximum stresses of 360 MPa for the third-floor case. And for the Interior frame, the first and third-floor cases, the reinforcement rupture at the same time after reaching 450 MPa tensile stresses.

Figure 22. Normal Stresses in Top Side RFT versus Time at column removal

- **Bottom Reinforcement**

  The normal stresses in bottom reinforcement for both Exterior and Interior frames are calculated as shown in Figures 23 and 24. For the bottom reinforcement at the sides, the reinforcement encountered compressive stresses due to the applied negative bending moment on the connection. For the Exterior frame, the maximum stresses of 200 MPa took place for the third floor and then stabilized.

  For the Interior frame, the bottom reinforcement failed for the first and third cases. The first floor failed earlier after reaching maximum stresses of 498 MPa. The fifth floor reached 360 MPa and then stabilized.

  At the column removal location, the bottom reinforcement of the Exterior frame reached maximum stresses of 360 MPa for the third-floor case. And for the Interior frame, the first and third-floor cases, the reinforcement rupture at the same time after reaching 450 MPa tensile stresses.

Figure 23. Normal Stresses in bottom RFT versus Time at sides
Figure 24. Normal Stresses in bottom RFT versus Time at column removal

**Column axial forces and Rotations**

For the Exterior frame, the increase in the axial forces in surrounding columns due to column removal is observed as follows; 82% increase in the first-floor case, 46% for the third-floor case and 23% for the fifth floor. For the Interior frame, no increase is calculated due to its collapse. Column rotations are calculated for the Exterior frame case, and all cases met the UFC rotation limits.

5.1.2. Edge Column Removal

The second part of the case study is the edge column removal from the exterior and the interior frames. The same parameters are studied.

**Failure Pattern**

For the exterior frame, the frame sustained the column removal for the three cases, on the contrary of the interior frame cases as shown in Figure 25.
**Beam Deflection**

The displacement is recorded at the location of the column removal. For the exterior frame, deflections vary from 86 mm for the first floor, 71 mm for the third floor and 324 mm for the fifth floor as shown in Figure 26. For the Interior frame, the three cases failed to resist the column removal.

**Connection Shear force**

The connection shear force after column removal is obtained for both cases. For the Exterior Frame, the maximum applied shear force in the connection reached 70 ton, 63 tons for the third floor and 71 ton for the last floor column removal at the removed column connection. For the Interior Frame, a maximum shear of 89.23 ton took place. For the Interior frames, shear forces are obtained at maximum values before the frame loses its stability. The maximum shear applied took place on the third floor with a value of 121.3 tons.

![Figure 26. Deflection versus Time](image)

![Figure 27. Shear Force in Connection](image)
Beam Axial Force

In the Exterior frame, the girder reached maximum compression of 400 KN and then sustained the column loss for the first floor, 350 KN for the third floor and zero force for the fifth floor.

In the Interior frame, the girder subjected to high compressive stresses of 500 KN for both first and third-floor cases and then shear failure took place as shown in Figure 28. For the last floor, no compression arching phenomenon took place, and the beam failed due to high negative bending moments resulted from load redistribution due to column loss.

Normal Stresses in reinforcement

The reinforcement locations studied are taken at the column removal and the side connection. For each side, top and bottom reinforcement are rerecorded.

- Top Reinforcement

In the Exterior frame, the fifth floor reached maximum stresses of 500 MPa higher than the first and third-floor cases and then stabilized. For the reinforcement at the column removal, the bars encountered compressive stresses due to the change in the structural system. Each case encountered different compressive stresses as shown in Figure 29. The maximum compressive stresses of 100 MPa, 120 MPa, and Zero for first, third and fifth-floor column removal took place.

In the Interior frame, at the side connection, for the three cases, the reinforcement resisted high tensile stresses till the rupture of reinforcement. The fifth-floor case reinforcement rupture took place earlier than the other two cases.

At the column removal connection, the reinforcement for the first and third floors encountered maximum compressive stresses 360 MPa. For the fifth floor, the top reinforcement didn't sustain any stress.
• Bottom Reinforcement

The normal stresses in bottom reinforcement for both exterior and interior frames are calculated as shown in Figures 31 and 32. For the bottom reinforcement at the sides, the reinforcement encountered compressive stresses due to the applied negative bending moment on the connection. For the Exterior frame, the maximum stresses of 340 MPa took place for the fifth floor and then stabilized. For the Interior frame, the bottom reinforcement failed for all cases with maximum stresses of 50 MPa.

At the column removal location, the bottom reinforcement of the Exterior frame reached maximum stresses of 350 MPa for the first-floor case. And for the Interior frame, the first and third-floor cases, the reinforcement rupture at the same time after reaching 490 MPa tensile stresses.

Figure 30. Normal Stresses in Top RFT versus Time at column Removal

Figure 31. Normal Stresses in bottom RFT versus Time at sides

Figure 32. Normal Stresses in bottom Side RFT versus Time at Column Removal
**Column axial forces and Rotations**

The percentage increase in axial force is calculated for the column in the middle span on the first floor. For the Exterior frame, the increase in the axial forces is 162% for the first-floor case, 100% for the third-floor case and 25% for the fifth floor. For the Interior frame, no increase is calculated due to its collapse. Column rotations are calculated for the Exterior frame case, and all cases met the UFC rotation limits.

6. Conclusion

Since the codes and regulations are not concerned about the connection behavior but the whole structural response, an investigation is conducted on special moment Column-Girder connection behavior, monolithically cast, against different column removal scenarios. The case study adopted is studied in detail. The failure pattern, internal stresses for the reinforcement, deflection of the main structural elements and internal forces in beams, columns and beam-column connection are extracted, and the following is concluded:

For the Exterior frame, no collapse is encountered due to many parameters. The frame location and load application are one of the main parameters that affected the frame behavior. The Vierendeel action effect in the first and third-floor cases is recognized compared to the fifth-floor case; this took place in the deflection of the girder as well as the normal stresses in the top and bottom reinforcement. The connection shear capacity specified by the ACI, which is already considered in the design, proved to resist the applied shear forces resulting from the redistribution of forces due to the column removal. Besides all this, there is a frame action that took place in cases of the third and fifth floors. The girders under the removed column floor worked as a tie member in resisting the lateral displacement caused by the compression arching phenomenon.

For the Interior frame, the applied loads are heavier compared to the Exterior frame. The failure is initiated by shear failure due to a high shear force followed by a rupture in the top reinforcement in the connections at the sides of the removed column. No collapse took place in the fifth-floor case, this is explained due to the absence of the effect of the upper floor that increased the applied loads on the 20-meter girder span.

For the edge column removal scenario, the girder acted as a cantilever with a span of 10 meters after the removal.

For the Exterior frame, the loads affected the frame behavior. No collapse occurred. The design nature of the frame leads the girder at the removal location to act as a broken frame girder with the upper column. This behavior took place due to the design nature of the Special moment connections that allowed the cantilever girder to work with the upper column as one unit. This redistribution along with the Vierendeel action helped to resist failure due to column loss.

For the Interior frame, the failure took place due to the heavy loads along with the cantilever action of the beam. The main cause of the failure is the high bending moment resulted from column removal. The Vierendeel action played a role in increasing the resistance of the structural components in resisting the failure in the first and the third-floor cases on the contrary to the fifth floor.

As a result, special moment connection could resist the column removal under light loading conditions; however, when the loads increase, some design considerations need to be suggested to resist either the excessive shear forces/bending moment to contribute with the other parameters that helped to withstand the failure in Exterior frame case. Experimental testing is not enough to study the structural behavior due to column loss due to the different parameters that are very difficult in measuring and controlling in the laboratories compared to the numerical methods used. 3D modeling and structure testing under column removal scenarios is a must, to study the structural integrity in all directions and to give a real indication on the structure behavior in progressive collapse.

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8. Conflicts of Interest

The authors declare no conflict of interest.

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Assessment of Moisture Susceptibility for Asphalt Mixtures Modified by Carbon Fibers

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Abstract

Moisture induced damage in asphaltic pavement might be considered as a serious defect that contributed to growth other distresses such as permanent deformation and fatigue cracking. This paper work aimed through an experimental effort to assess the behaviour of asphaltic mixtures that fabricated by incorporating several dosages of carbon fiber in regard to the resistance potential of harmful effect of moisture in pavement. Laboratory tests were performed on specimens containing fiber with different lengths and contents. These tests are: Marshall Test, the indirect tensile test and the index of retained strength. The optimum asphalt contents were determined based on the Marshall method. The preparation of asphaltic mixtures involved three contents of carbon fiber namely (0.10%, 0.20%, and 0.30%) by weight of asphalt mixture and three lengths including (1.0, 2.0 and 3.0) cm. The results of this work lead to several conclusions that mainly refer to the benefits of the contribution of carbon fibers to improving the performance of asphalt mixtures, such as an increase in its stability and a decrease in the flow value as well as an increase in voids in the mixture. The addition of 2.0 cm length carbon fibers with 0.30 percent increased indirect tensile strength ratio by 11.23 percent and the index of retained strength by 12.52 percent. It is also found that 0.30 % by weight of the mixture is the optimum fiber content for the three lengths.

Keywords: Asphalt; Moisture Damage; Carbon Fiber; Compressive Strength; Indirect Tensile Strength.

1. Introduction

Asphalt pavements deteriorate over time because of the combined effect of traffic loading and the environment. The service life of asphalt pavements can even be cut short if quality materials are not used in the hot mix asphalt (HMA) design and manufacturing [1]. In recent years, researchers have used different types of additives to improve the performance of the asphalt mixture. Scientists and engineers are permanently trying to improve properties of asphalt mixtures, such as their stability and durability, by incorporating new additives either in the bitumen or in the asphalt mixture [2, 3]. The addition of polymers is a common method applied to modify the binder, although different types of fibers have been evaluated [4]. However, it has been claimed that among various modifiers for asphalt, fibers have gotten much attention due to their improving effects [5]. In asphalt concrete (AC), fibers have been added to prevent draindown or raveling of porous asphalt and stone matrix asphalt, and to improve resistance to cracking and rutting [6]. Adding fibers to the binder or the bituminous mixtures ensures their stability and mechanical strength [7].

The types of fibers that have been investigated to date are polymeric fibers (polyester, polypropylene, polyacrylonitrile), organic fibers (cellulose, lignin, date-palm, oil-palm), mineral fibers (asbestos, rock wool), waste

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fibers (nylon, scrap tire, textile), and other fibers (glass, carbon, steel) [8]. Fibers provided this material with a higher modulus, resistance, durability, and deformation, and thus with more ductile behavior [9].

Artificial fibers are usually preferred, as natural ones are highly water absorbent [10]. Fiber's effects on improving the engineering properties of the asphalt mixture have been recognized by previous research. Some researchers have reported that fiber increases the optimum asphalt content in the mixture design and prevents asphalt leakage due to its absorption of asphalts [11], improves moisture susceptibility and reduces the reflective cracking of AC mixtures and pavements [12]. As a reinforcing material, the primary function of fibers is to provide an additional tensile strength in the resulting composite. As a kind of modification, carbon fiber was added to the asphalt material to investigate the performance of many researchers.

Jahromi and Khodaii (2008) studied the characteristics and properties of carbon fiber reinforced asphalt mixtures. It found that the addition of carbon fibers increases the mix’s stability and voids in the mix while decreases flow value. They also found that the addition of fibers can improve the fatigue life and permanent deformation of the mixtures [4].

Liu and Wu (2011) found the carbon fiber could increase the Marshall stability, residual stability, and rutting dynamic stability. Also, measuring the mechanical and electrical properties of graphite and carbon fiber modified asphalt concrete [13]. Khattak et al. (2012) believed that the carbon nanofibers modified asphalt binder could increase the complex shear modulus of asphalt binder and fatigue life of asphalt mixture [14].

Cleven (2000) investigated the behavior of carbon fiber modified asphalt mixtures. Carbon fibers were found to create improvements in high temperature and low temperature behavior. HMA samples containing 0.5% to 0.8% and fiber length taken from 2.54 cm to between 0.2mm and 0.65mm. Resilient modulus, creep compliance, beam fatigue, and indirect tensile tests were used to evaluate the effect of carbon fiber on HMA. The results showed that carbon fiber modified HMA was stiffer and performed better against permanent deformation. The modified HMA also showed improved tensile strength; however, no significant difference was observed in fatigue resistance [15].

Moghadas et al. (2014) developed an approach to mix carbon fibers and bitumen which guarantees uniform fiber distribution. Subsequently, to find out the best set of fiber lengths and a dose of usage aimed at fortifying asphalt concrete, Marshall's stability and fatigue property of carbon fiber-reinforced asphalt concrete were investigated. Then, indirect tensile stiffness modulus and fatigue properties under different stresses and permanent deformation of modified and unmodified samples at two different temperatures (35°C and 60°C) were studied. Comparing the obtained results indicated that the addition of carbon fibers to the asphalt concrete considerably increases the mechanical performance, which benefits all the corresponding fields involved such as repair and maintenance [16].

Doo-Yeol et al. (2019) investigated the effect of carbon-based materials, i.e., carbon fibers (CFs), carbon nanotubes (CNTs), and graphite nanofibers (GNFs), on the mechanical properties of asphalt concrete. For this, 0.5% CF, CNT, and GNF, and 1.0% CF were incorporated, and plain asphalt concrete was also considered for comparison. Test results indicated that adding the carbon nanomaterials, i.e., CNTs and GNFs, was more effective in improving the Marshall stability, indirect tensile strength, and dynamic stability, and reducing the porosity, compared to adding macro CFS. However, the flexural performance of asphalt concrete was more efficiently enhanced by adding the CFs relative to CNTs and GNFs [17].

Khabiri and Alidadi (2017) studied the properties of reinforced asphalt mixtures with carbon and glass fibers. To evaluate the effect of carbon fiber and glass on asphalt mixtures. Physical performance, Marshal stability, and indirect tensile tests were performed by international standards for samples with fiber and without fiber by dry method. Three contents of carbon fiber (0.1, 0.2 and 0.3) % by weight of asphalt mixture were used. The results of the study indicated that the behavior of fracture for fibrous samples was somewhat different from the fracture of non-fibrous samples [18].

Abtahi et al. (2010) investigated the advantages of using random-inclusion fibers in flexible pavements as fiber-reinforced asphalt-concrete. They found that fibers change the viscoelasticity of mixture; moisture susceptibility and reduce the reflective cracking of asphalt mixtures and pavements. These properties were separately discussed for different kinds of fibers including Polypropylene, Polyester, Asbestos, Cellulose, Carbon, Glass and Nylon. Polyester was chosen over polypropylene because of its higher melting point. In addition, methods of sample preparation and executive problems conversed. Therefore, it was found that there are two potential methods for the introduction of the fibers: the wet process (addition of fibers to the asphalt) and the dry process (addition of fibers to the dry aggregate). They found that the dry one is preferred over the other [19].

Liu and Wu (2011) studied the mechanical and electrical properties of modified graphite and modified carbon fiber were measured by indirect tensile test. Experimental results suggest that mechanical properties of the asphalt mixture are affected by the addition of a conductive component such as Graphite and carbon fiber, when graphite content increased from 0 to 22% volume, and Marshall stability decreased from 12.8 kN to 9.43 kN and when the carbon fiber content increased from 0 to 2% Marshall stability increased from 12.8 kN to 13.5 kN [13].
Yang et al. (2006) studied the rutting resistance for cellulose and polyester fiber-reinforced asphalt by using both the Circular Road Track and Hamburg Wheel Tracking Device tests. The obtained results from two tests showed that the rut depth can be significantly improved by adding fibers, but polyester fibers are better and improve the rutting resistance than that of cellulose fibers [20].

Liu et al. (2010) used steel fibers as the conductive filler to enhance the electrical conductivity of porous asphalt and it is found that long fibers with a small diameter are better than short fibers with a bigger diameter to make higher electrical conductivity in porous asphalt [21].

Ismael and Al-Taher (2015) studied the role of polyester fibers as reinforcement in the process of paving materials improvements. Marshall Stability, Indirect Tensile Strength (ITS) and compressive strength are studied. The indication of moisture damage resistance represented by the "Indirect Tensile Strength Ratio" and "Index of Retained Strength" has been calculated. Also, the flexural bending test of beam specimens has been carried out. They are used three percentages (0.25, 0.50 and 0.75 by weight of mixture with the two lengths (6.35 and 12.70 mm). It was found that (0.5) % by weight of the mixture is the optimum fiber content for the two lengths [22].

Chen et al. (2009) founded the volumetric and mechanical characteristics obtained by the fiber-reinforced asphalt mixture design method. The obtained results indicated that optimum bitumen content, air void, volume of mineral aggregate and Marshall Stability value increased, while unit weight decreased with the addition of fiber [23].

Mehrez and Karim (2010) explained that reinforcing generally includes the addition of materials with certain required properties to other materials that do not contain these properties. Fiber reinforcement was used to carry tensile loads, improve pavement resistance to some distress as well as to prevent the formation and spread of cracks [24].

The primary objective of this work is to evaluate the effect of the addition of different length and content of carbon fibers on Marshall properties and moisture susceptibility of asphalt mixtures considering the values of the Index of Retained Strength and the Tensile Strength ratio to show the reinforcement effect to control the harmful effect of moisture.

2. Research Methodology

The first phase of the laboratory work is represented by asphalt binder conventional test for a virgin asphalt. The second phase includes the design of the asphalt mixture with and without carbon fiber by using Marshall method and obtaining the optimum asphalt content. Furthermore, the stability and flow value in addition to the volumetric properties were determined to satisfy the requirement of Iraqi specification. Following this phase, the compressive strength and the indirect tensile strength value were determined to find the Index of retained strength (IRS) and tensile strength ratio (TSR) to find out the best content and length of carbon fiber was added to resist moisture damage as shown in Figure 1.

2.1. Materials and Methods

Asphalt Cement

Asphalt binder (40-50) penetration grade was used in this study which was obtained from Al-Durrah Refinery. All test results of asphalt binder satisfied the requirement of the State Commission of Roads and Bridges (SCRB R/9, 2003) specification [25]. Table 1, exhibits the physical properties of asphalt cement which was used in this research.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Penetration (25°C, 100 gm, and 5sec)</td>
<td>44(0.1mm)</td>
<td>40 - 50</td>
<td>D-5</td>
</tr>
<tr>
<td>Ductility, (25 °C and 5cm/minute),</td>
<td>151 (cm)</td>
<td>≥ 100</td>
<td>D-113</td>
</tr>
<tr>
<td>Softening point, (Ring and Ball)</td>
<td>50 °C</td>
<td>------</td>
<td>D-36</td>
</tr>
<tr>
<td>Flash point,(Cleveland open Cup)</td>
<td>280 °C</td>
<td>&gt;232</td>
<td>D-92</td>
</tr>
<tr>
<td>Specific Gravity @ 25°C</td>
<td>1.03</td>
<td>------</td>
<td>D-70</td>
</tr>
<tr>
<td>Retained penetration, of original, (%)</td>
<td>61(0.1mm)</td>
<td>&gt;55</td>
<td>D-5</td>
</tr>
<tr>
<td>Ductility (25°C and 5cm/minute)</td>
<td>89 (cm)</td>
<td>&gt;25</td>
<td>D-11</td>
</tr>
</tbody>
</table>

Table 1. Physical properties of asphalt cement
Coarse and Fine Aggregates

The coarse 12.5 mm nominal aggregate maximum size brought from Al-Nibaa quarry was used in preparation as asphalt mixture. The gradation of fine aggregate is between No.4 sieve size (4.75 mm) and No.200 sieve size (0.075 mm). The aggregates were tested for physical properties and Table 2 presents the test results.

<table>
<thead>
<tr>
<th>Property</th>
<th>ASTM Designation No.</th>
<th>Coarse Aggregate</th>
<th>Fine Aggregate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bulk Specific Gravity</td>
<td>C-127 &amp; C-128</td>
<td>2.618</td>
<td>2.576</td>
</tr>
<tr>
<td>Percent Water Absorption</td>
<td>C-127 &amp; C-128</td>
<td>0.435</td>
<td>0.562</td>
</tr>
<tr>
<td>(Los Angeles Abrasion) %</td>
<td>C-131</td>
<td>18</td>
<td>..........</td>
</tr>
</tbody>
</table>

Mineral Filler

The filler is a material passing sieve No.200 (0.075 mm). It was decided to use limestone dust as filler in preparing the asphalt mixture due to its accessibility and relatively lower cost. It was brought from a lime factory in Karbala governorate. The physical properties of the mineral filler are listed in Table 3.
Carbon Fiber (CF)

Carbon fiber was brought from a Sika company with the cost of the roller (20) $ per meter width 50 cm. It was chosen because of its higher tensile strength property and higher melting point. Three lengths (1.0, 2.0 and 3.0) cm and three percentages (0.10, 0.20 and 0.30) % by weight of asphalt mixture were selected. The physical characteristics of carbon fibers are shown in Table 4.

<table>
<thead>
<tr>
<th>Test Properties</th>
<th>Typical Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nominal thickness (mm)</td>
<td>0.167</td>
</tr>
<tr>
<td>Fiber Length (mm)</td>
<td>Can be produced by any length</td>
</tr>
<tr>
<td>Color</td>
<td>Black</td>
</tr>
<tr>
<td>Density gm/cm³</td>
<td>1.82</td>
</tr>
<tr>
<td>Tensile Strength (N/mm²)</td>
<td>40000</td>
</tr>
<tr>
<td>Elongation-at-Break, %</td>
<td>1.7</td>
</tr>
<tr>
<td>Tensile Modulus of elasticity(kN/mm²)</td>
<td>225</td>
</tr>
<tr>
<td>Base</td>
<td>Polycrylonitrile</td>
</tr>
<tr>
<td>Temperature of Carbonization</td>
<td>1400 °C</td>
</tr>
</tbody>
</table>

2.2. Selection of Aggregates Gradation

The selection of the aggregates in this work is the following (SCRB R/9, 2003), [32] for wearing course with a nominal maximum size of aggregate of 12.5 mm. Figure 2 presents the gradation of the selected aggregates for the wearing course.

3. Preparation of Marshall Mixtures

The aggregates are separated into the desired size and recombined with the mineral filler to meet the required gradation according to the (SCRB, 2003) for wearing course. The dry mixing method was implemented, and by visual
comparison, it was realized that the dry mixing process was more practical in dispersing fibers and employing fibers through the mixture. As a result, the dry procedure was chosen for use. The carbon fiber is blended with aggregates for 40-60 seconds. The aggregates are then heated to a temperature (155-165 °C) before mixing with asphalt cement which has already been heated to a temperature that produces a kinematic viscosity of (170 ± 20) centistokes (up to 163 °C as an upper limit). The asphalt cement was then added to the preheated aggregate to reach the required amount of asphalt content. Mix asphalt and aggregate in a mixing bowl by hand on the hot plate for (3-4) minutes until the asphalt adequately covers the surface of the aggregate and fiber as shown in Figure 3.

![Figure 3. preparation of asphalt mixtures](image)

4. Indirect Tensile Strength Test (ITS)

This test was performed according to the method described in ASTM D-6931 and explained in [26]. The same Marshall mold dimensions have been utilized with the total number equal to 21 specimens. They left to cool at room temperature for 24 hours and then placed in an air bath at 25 °C for 4 hours to bring them to test temperature. The loading strips were placed and the load was applied at a strain rate of 50 mm/min as shown in Figure 4. Three specimens for each mix combination were tested and the average results were reported. The indirect tensile strength in kPa was then calculated as follows.

\[
I.T.S = \frac{2P}{\pi t D}
\]

\[
TSR = \frac{I.T. S_s}{I.T. S_d}
\]

Where; P: Ultimate applied load required to fail specimen (N); t: Thickness of the specimen (mm); D: Diameter of specimen (mm); I.T. Ss: indirect tensile strength for soaked specimens (kPa); I.T. Sd = indirect tensile strength for dry specimens (kPa).

![Figure 4. Indirect Tensile Strength Specimen](image)

5. Compressive Strength Test

This method covers the measurement of the loss of cohesion resulting from the action of water on compacted bituminous mixtures. Specimens 4.0 in (101.6 mm) in diameter and 4 in. (101.6 mm) in height were prepared. A set of six specimens prepared for this purpose. Three specimens were tested for compressive strength after storing in an air bath at 25 °C for about 4 hours. The other three specimens were placed in a water bath for 24 hours at 60 °C, then transferred to a water bath, and maintained at 25 °C for 2 hours before testing for compressive strength as shown in Figure 5. The index of retained strength is calculated as follows in accordance with ASTM D-1075-07 [27], which should be a minimum of 0.7 (or 70%) as adopted by (SCRB/R9, 2003) [25].
6. Results and Discussion

6.1. Marshall Test

The Marshall test was performed to determine the optimum asphalt content for different mixtures groups. The control mix contains (0%) CF achieved 4.90% of the asphalt content (depending on weight of the total mixture). This percentage increases slightly when the carbon fiber added to the asphalt mixture, three percentages of carbon fiber contents (0.10, 0.20 and 0.30) % (by weight of asphalt mixture) with three fiber lengths (1.0, 2.0 and 3.0) cm spread during mixing. The optimum asphalt content for mixture with fiber percentages of (0.1, 0.2 and 0.3) are obtained as 5.0, 5.15 and 5.3 respectively for a length of 1.0cm. The optimum asphalt content for mixture with fiber percentages of (0.10, 0.20 and 0.30) are also obtained as 5.1, 5.25 and 5.4 % respectively for a length of 2.0 cm. The optimum asphalt content for mixture with fiber percentages of (0.10, 0.20 and 0.30) are also obtained as 5.23, 5.3 and 5.5 % respectively for a length of 3.0 cm. Table 5 shows the results of Marshall test. The optimum content of asphalt for all of the fiber mixtures had a higher optimum percentage of asphalt content than the control mixture, for length 1.0 cm OAC was higher than control mix by (2.0, 5.1 and 8.16) % for fiber content (0.1, 0.2 and 0.3) % respectively, for 2.0 cm OAC was higher than control mix by (4.08, 6.12 and 10.2) % for fiber content (0.10, 0.20 and 0.30) % respectively and for length 3.0 cm OAC was higher than control mix by (6.7, 8.16 and 12.24) % for fiber content (0.10, 0.20 and 0.30) % respectively. This increase dependent on the absorption and surface area of the fibers and thus not only affected by different concentrations of fibers but also by the length of different fibers. Adding fiber requires more asphalt to wrap on its surface because of it’s relatively define by surface area and absorption light asphalt components. Marshall test showed that the stability of mixtures containing fibers was higher than the stability of control mixtures. Specimens containing fiber length (1.0 cm and 2.0 cm) had higher stability values than the control mixtures by (19.28, 27.18 and 31.5) % and (26.7, 38.9 and 51) % for (0.10, 0.20 and 0.30) % CF respectively, while specimens containing (0.10, 0.20 and 0.30) % by weight of asphalt mixture with fiber length 3.0 cm had higher stability values than the control mixtures by (21.2, 17.36 and 10.17) % respectively. The increase in the stability values as the fiber increase in content and length to a certain percentage, then begin to decrease. These results could be attributed to fibers adhesion and networking effects. A large amount of fiber in the mixture produces low contact points between the aggregates, leading to less stability. The decrease shows that the fiber length has little effect on the stability of the mixture. Fibers also form spatial networking to stabilize and strength mixture. Therefore, excessive fiber may do not disperse uniformly, while coagulating together to form a weak Point inside the mixture. As a result, Marshall stability is decreasing in high fiber content.

The flow values were lower than the control mix for fiber length (1.0 cm and 2.0 cm) for mixes containing (0.10, 0.20 and 0.30) % by weight of asphalt mixture. Flow value decrease by (3.12, 9.37 and 9.37) % for fiber content (0.1, 0.2 and 0.3) % respectively, for 2.0 cm OAC was higher than control mix by (4.08, 6.12 and 10.2) % for fiber content (0.10, 0.20 and 0.30) % respectively and for length 3.0 cm OAC was higher than control mix by (6.7, 8.16 and 12.24) % for fiber content (0.10, 0.20 and 0.30) % respectively. This increase dependent on the absorption and surface area of the fibers and thus not only affected by different concentrations of fibers but also by the length of different fibers. Adding fiber requires more asphalt to wrap on its surface because of it’s relatively define by surface area and absorption light asphalt components.

The flow values for mixes containing (0.10, 0.20 and 0.30) % by weight of asphalt mixture with fiber length 3.0 cm shows an increase in flow about (6.25, 15.62 and 31.25) % over the control mix. This increase in flow values can be due to excessive asphalt content of induced fiber mixtures. The presence of further bitumen makes the aggregates “float” within the mix leading to higher flow value. The effect of fiber length on the flow is clearer of stability.

The bulk density of the asphalt mixture decreases with the increase in fiber content and length. The mixtures with (1.0, 2.0 and 3.0 cm) long have a lower density than the control mixtures. Specimens containing (0.10, 0.20 and 0.30) % CF by weight of asphalt mixture with fiber length 1.0cm had lower bulk density than the control mixture by (0.3, 0.47 and 1.96) % respectively, while specimens containing (0.10, 0.20 and 0.30) % of CF with 2.0 cm long had lower bulk density than the control mixture by (1.28, 1.75 and 2.52) % respectively and specimens containing (0.10, 0.20 and 0.30) % of CF with 3.0 cm long had lower bulk density than the control mixture by (2.12, 2.68 and 3.12) % respectively.
% of CF with 3.0 cm long had lower bulk density than the control mixture by (2.22, 2.85 and 3.5) % respectively. This is related to the higher optimum asphalt contents and fibers at high percentage are distributed less homogeneously when mixed, which increase the formation of fiber blends. The increase of OAC requires more compaction effort to achieve the same density due to the relatively lower specific gravity of asphalt than that of mineral aggregate or reduces the density of asphalt mixture at the same compaction effort. As a result, at the same compaction effort (75 blows on both sides of the Marshall sample), adding fiber reduces the bulk density of asphalt mixture. Therefore if it needs to improve bulk density in the field construction more compaction efforts than the ordinary mixture will be essential.

The specimens containing no fibers had lower air void contents than the mixtures containing carbon fibers. It was also observed that the specimens made with 0.30% fiber contents had higher air void contents than the specimen containing (0.10 and 0.20)% fiber contents. Specimens containing (0.10, 0.20 and 0.30)% by weight of total mixture with fiber length 1.0 cm had higher air voids than the control mixture by (4.63, 8.7 and 16)% respectively while specimens containing (0.10, 0.20 and 0.30) % of fiber with 2.0 cm long had higher air voids than the control mixture by (8.11, 18.8 and 21.73)% respectively and specimens containing (0.10, 0.20 and 0.30) % of fiber with 3.0cm long had higher air voids than the control mixture by (21.73, 30.43 and 42)% respectively. Also, specimens containing (0.10, 0.20 and 0.30)% CF with 3.0 cm long had lower bulk density than the control mixture by (2.22, 2.85 and 3.5)% respectively, as shown in Figure 6. Results show that the mean wet I.T.S. values of all the fiber mixtures were greater than the control mixtures. The mean wet I.T.S. values of the specimens containing CF with 2.0 cm long were higher than control mixture by (18.6, 26.26 and 37.91)% respectively and the mixtures with (0.10, 0.20, and 0.30)% fiber content (by weight of asphalt mixture) with 3.0 cm length were higher than control mixture by (17.9, 15.16 and 5.6)% respectively as shown in Figure 7. It is observed that as the fiber content increases, the tensile strength increases slightly. This observation exhibits that the effect of fiber length on strength is significantly large so that the combined effect of fiber content and fiber length leads to only a slight increase in the strength.

An increase in fiber content and length in the mixture followed an increase in the (V.M.A) due to the decrease of bulk density (lower bulk density results in higher VMA). This property is significant in so far as the pavements of hot regions are concerned because asphalt may be prone to bleeding and increasing void ratio could prevent bleeding by providing more spaces for the binder to move into.

### Table 5. Marshall Test Results

<table>
<thead>
<tr>
<th>Fiber length cm</th>
<th>CF (%) by wt. of Asphalt mixture</th>
<th>O.A.C., (%) by wt. of mix.</th>
<th>Stability (kN)</th>
<th>Flow (mm)</th>
<th>Bulk Density (gm/cm³)</th>
<th>Air Voids (%)</th>
<th>V.M.A (%)</th>
<th>V.F.A (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>control</td>
<td>0</td>
<td>4.9</td>
<td>8.35</td>
<td>3.2</td>
<td>2.344</td>
<td>3.45</td>
<td>15.18</td>
<td>78</td>
</tr>
<tr>
<td>1.0</td>
<td>0.10</td>
<td>5.0</td>
<td>9.96</td>
<td>3.1</td>
<td>2.337</td>
<td>3.61</td>
<td>15.5</td>
<td>76.69</td>
</tr>
<tr>
<td></td>
<td>0.20</td>
<td>5.15</td>
<td>10.62</td>
<td>2.9</td>
<td>2.333</td>
<td>3.75</td>
<td>15.7</td>
<td>76.1</td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>5.3</td>
<td>10.98</td>
<td>2.9</td>
<td>2.298</td>
<td>4</td>
<td>17.2</td>
<td>75.4</td>
</tr>
<tr>
<td>2.0</td>
<td>0.10</td>
<td>5.1</td>
<td>10.58</td>
<td>3</td>
<td>2.314</td>
<td>3.73</td>
<td>16.45</td>
<td>76</td>
</tr>
<tr>
<td></td>
<td>0.20</td>
<td>5.2</td>
<td>11.6</td>
<td>2.8</td>
<td>2.303</td>
<td>4.1</td>
<td>16.8</td>
<td>75</td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>5.4</td>
<td>12.62</td>
<td>2.7</td>
<td>2.285</td>
<td>4.2</td>
<td>17.72</td>
<td>74.8</td>
</tr>
<tr>
<td>3.0</td>
<td>0.10</td>
<td>5.23</td>
<td>10.12</td>
<td>3.4</td>
<td>2.292</td>
<td>4.2</td>
<td>17.4</td>
<td>74.7</td>
</tr>
<tr>
<td></td>
<td>0.20</td>
<td>5.3</td>
<td>9.8</td>
<td>3.7</td>
<td>2.277</td>
<td>4.5</td>
<td>18</td>
<td>72.5</td>
</tr>
<tr>
<td></td>
<td>0.30</td>
<td>5.5</td>
<td>9.2</td>
<td>4.2</td>
<td>2.262</td>
<td>4.9</td>
<td>18.6</td>
<td>72.3</td>
</tr>
</tbody>
</table>

### 6.2. Indirect Tensile Strength Test Results

The tensile strength ratio (TSR) was used to predict the moisture sensitivity of the mixtures. The recommended TSR limit of 0.8 is used to distinguish between moisture sensitive mixtures and moisture resistance mixtures as adopted by (ASTM D-4867). High TSR values indicate that the mixture will work best to resist moisture damage. The dry I.T.S. values of the mixtures containing fibers were higher than the control mixtures. The values of dry I.T.S. were higher for 2.0 cm than (1.0 and 3.0 cm) long. Specimens containing (0.10, 0.20, and 0.30) % CF with 1.0 cm long have higher dry I.T.S. values than the control mixtures by about (7.57, 12.5 and 21.9)% respectively, also specimens containing (0.10, 0.20, and 0.30)% CF with 2.0 cm long have higher dry I.T.S. values than the control mixtures by about (13.64, 16.55, and 24)% respectively and specimens containing (0.10, 0.20, and 0.30)% CF with 3.0 cm long have higher dry I.T.S. values than the control mixtures by about (11.35, 10.56, and 3.52)% respectively as shown in Figure 6. Results showed that the mean wet I.T.S. values of all the fiber mixtures were greater than the control mixtures. The mixtures with (0.10, 0.20, and 0.30)% fiber content (by weight of asphalt mixture) with 1.0 cm length were higher than control mixture by (10.76, 18.35, and 32.19)% respectively, also the mixtures with (0.10, 0.20, and 0.30)% fiber content (by weight of asphalt mixture) with 2.0 cm length were higher than control mixture by (17.8, 26.26 and 37.91)% respectively and the mixtures with (0.10, 0.20, and 0.30)% fiber content (by weight of asphalt mixture) with 3.0 cm length were higher than control mixture by (17.9, 15.16 and 5.6)% respectively as shown in Figure 7. It is observed that as the fiber content increases, the tensile strength increases slightly. This observation exhibits that the effect of fiber length on strength is significantly large so that the combined effect of fiber content and fiber length leads to only a slight increase in the strength.
It should be noted that the increase in dry I.T.S may be due to the high tensile modulus of elasticity and lower ability of extension of carbon fibers. In the proposed sample, the fibers have a random orientation in a different direction which strongly binds particles inside the matrix and prevents them from moving, thus making the mixture stiffer. Fiber carries some tensile loads when bitumen fiber mastics are tested under tension, the stress is transmitted from the matrix to the fiber. Fiber carries some tensile loads part of the tensile stress can be achieved by fiber, so the wet tensile strength increases with increased fiber concentration and length to a certain extent then begin to decrease. This increase associated with the fact that the presence of fibers increases the strength of the mixture due to the interlocking phenomenon. The networking between fibers and bitumen seems to prevent the penetration of water into the mixture. This plays an important role in increasing wet tensile strength making the mixture more resistant to moisture damage. The use of an appropriate fiber type and content has an important effect on the tensile strength of the asphalt mixture.

![Figure 6. Effect of CF and length on I.T.S of dry specimens](image6.png)

T.S.R values of control mixtures were significantly lower than that of fiber mixtures. T.S.R values of 1.0 cm length is increased by about (2.99, 5.11 and 8.48) for (0.10, 0.20 and 0.30)% fiber content respectively than the control mixture while T.S.R values of 2.0 cm length is increased by about (3.62, 8.36 and 11.23)% for (0.10, 0.20 and 0.30)% fiber content respectively than the control mixture and T.S.R values of 3.0 cm length is increased by about (6.24, 4.74 and 1.87)% for (0.10, 0.20 and 0.30)% fiber content respectively than the control mixture as shown in Figure 8.

![Figure 7. Effect of CF and length on I.T.S of wet specimens](image7.png)
6.3. Compressive Strength Test Results

According to ASTM D-1074 and D-1075, (SCRB, 2003) indicated that the acceptable value of the index of retained strength is (70%) or higher, so any mixture has an I.R.S. below this value is considered susceptible to water damage. The dry compressive strength values of the mixtures containing fibers were higher than the control mixtures. Specimens containing (0.10, 0.20 and 0.30) % carbon fibers (by weight of asphalt mixture) have higher compressive strength values than the control mixtures by about (2.4, 4.32 and 7.18) % for length 1.0 cm, (2.84, 8.83 and 10.43) % for length 2.0 cm and specimens with a length of 3.0 cm containing 0.10% CF were higher control mixture by (0.14) %, while the other percentage (0.20 and 0.30) % of CF (by weight of asphalt mixture) have lower compressive strength values than the control mixtures by (1.14 and 2.97) % respectively as shown in Figures 9 to 11.

The mean wet compressive strength values of all the fiber mixtures were greater than the control mixtures. The wet compressive strength values for mixtures with (2.0 cm) fiber length found to be higher than the mixtures with (1.0 and 3.0 cm) fiber length. The wet compressive strength value of (1.0 cm) fiber length increased by about (6.21, 11.3 and 16) %, for 2.0 cm fiber length is increased by about (9.4, 19 and 24) %. Specimens with (3.0 cm) fiber length content (0.1 and 0.2) % carbon fiber increased by about (5 and 1.77) % than the control mixture and the specimens containing (0.3) % fiber content (by weight of asphalt mixture) decrease by about (4.72) % than the control mixture as shown in Figures 9 to 11.

Dry and wet compressive strength values for 0.30% fiber content were higher than (0.10 and 0.20) % fiber content for length (2.0 cm). Higher wet compressive strength of fiber mixtures could be associated to the fact that presence of fibers with the bitumen producing network bonding which prevent the penetration of water to the mixture and this increases the strength of the mixture and make the mixture more resistant to moisture damage indicating that the use of carbon fibers decrease the moisture susceptibility of mixtures.

The values of the index of retained strength (I.R.S.) of control mixtures were significantly lower than that of fiber mixtures. The percentage and size of fibers had significant effects on I.R.S values. The mixtures with 2.0 cm long have higher I.R.S. values than mixtures with (1.0 and 3.0 cm) long. I.R.S values of 1.0 cm length is increased by about (3.8, 6.6 and 8.3) %, for 2.0 cm length is increased by about (6.51, 10 and 12.5) % for (0.10, 0.20 and 0.30) % carbon fiber and I.R.S values of 3.0 cm length is increased by about (5 and 2.9) % for (0.10 and 0.20) % fiber content respectively than the control value and decrease by about (1.76) % for 0.3% fiber content as shown in Figure 12. Higher I.R.S. refers to lower moisture susceptibility and good resistance to moisture damage. This indicates that 0.30% is the optimum fiber content and the increase in fiber length may not blend well with the mixture. Higher I.R.S. refers to lower moisture susceptibility and good resistance to moisture damage.
Figure 9. Effect of Fiber Percentages on Dry and Wet Compressive Strength for (1 cm Fiber Length)

Figure 10. Effect of Fiber Percentages on Dry and Wet Compressive Strength for (2 cm Fiber Length)

Figure 11. Effect of Fiber Percentages on Dry and Wet Compressive Strength for (3 cm Fiber Length)
7. Conclusions

- The addition of carbon fibers to the asphalt mixtures showed an improvement in Marshall properties. Stability values increased when CF increased by length and content to a certain limit and then decreased. Specimens containing 2.0 cm long fiber mixtures had higher stability values than the 1.0 cm and 3.0 cm long fiber. Flow value decreases when CF adding to the mixture. Air void and VMA increase, while bulk specific gravity decreases after adding carbon fibers or increasing fiber content in asphalt mixture.

- The tensile strength ratios for all mixtures containing carbon fiber were higher than those without fibers mixtures. This indicates that the use of carbon fibers decreased moisture susceptibility of asphaltic mixtures. The 0.30 % fiber content gives the highest value of the Tensile Strength Ratio for lengths 1.0 and 2.0 cm. T.S.R value increased by 8.48 % for the 1.0 cm fiber length and 11.23 % for the 2.0 cm fiber length while 3.0 cm fiber length recorded the highest increment in T.S.R with 0.10 % CF by about 6.24%.

- The Index of Retained Strength values increases for all mixtures containing fibers. This indicates that the use of carbon fibers decreased moisture susceptibility of asphaltic mixtures. The 0.30 % fiber content gives the highest value of Index of Retained Strength. For this content, the I.R.S. value increased by 8.3 percent for the 1.0 cm fibers length, 12.52 percent for the 2.0 cm fibers length. Excluding 0.30% with 3.0 cm fiber length reduced by about 1.76% of the control value.

- The 0.30 % carbon fiber with 2.0 cm fiber length leads to the best values of indirect tensile strength ratio and index of retained strength and exhibited better mechanical behavior than longer fibers, which may lead to the phenomenon of “balling” in the mix which lead to lose its positive effects.

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9. Conflicts of Interest

The authors declare no conflict of interest.

10. References


Hydrochemical Characterisation of Groundwater Quality: 
Merdja Plain (Tebessa Town, Algeria)

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Abstract

The objective of this work is to evaluate the physico-chemical quality of the groundwater of the Merdja plain and to determine the sources of mineralization. This quality is influenced by several environmental and anthropogenic factors such as geological context, climate, precipitation and interaction between groundwater and aquifers and human activities. A Principal Component Analysis (PCA) on samples taken from several wells spread over the entire Tebessa plain (Merdja) allowed us to detect two axes that explain 73.4% of the information. The first axis describes the variables related to mineralisation and the second one describes those related to agricultural activity. Multidimensional Positioning (MDS) confirmed the interaction of physico-chemical parameters between them and their influence on groundwater quality by highlighting three groups of wells according to their physico-chemical characteristics, particularly those containing high concentrations of nitrates. This contamination is mainly the result of spreading the fertilisers and wastes that are dumped into the plain without treatment. Salinization is the result of long-term interactions between groundwater and geological formations.

Keywords: El Medja Plain; Groundwater; Hydrochemical; Geological Formations; PCA; MDS.

1. Introduction

Tébessa region is located near the Algerian-Tunisian borders, in the eastern part of the Saharan Atlas, between the Tellian Atlas and the Saharian platform. According to Vila (2002) and Kowalski et al. (2002) this is a very neotectonized domain, which extends into extreme western Tunisia, belongs to the area of diapirs and grabens. Due to its recharge modalities, the water from the Tébessa aquifer presents a high risk of increasing its mineralization [1, 2]. Over the past few decades, the alluvial groundwater of Tébessa has undergone a rapid deterioration in the chemical quality of these waters [3-5]. Indeed, this phenomenon is simultaneously attributed to the geological context and the semi-arid climate. It is crossed with an over exploitation of the modest resource to meet the needs of drinking water supply and crop irrigation. However, the problem of groundwater pollution has been addressed in several studies focusing on the vulnerability of the aquifer to contamination by salts from saline soils and Triassic out crops, or to pollution by nitrates. This paper highlights only the most recent and complete studies.

In order to identify the processes of mineralization, its origin and evolution in time and space, Gouaidia et al. (2012) monitored water chemistry during two hydrological cycles covering the periods of low and high water. Results showed that mineralization processes are linked to the lithology of the aquifer presented, mainly by evaporites and carbonates. Water-rock interaction phenomena are at the origin of the spatial variation of groundwater geochemistry. On the other
hand, climatic conditions are responsible for variations in concentrations due to precipitation (dissolution) and temperature (evaporation) [6].

Rouabhia et al. (2009) have used data obtained from the same area through a sampling period (December 2008) and indicates that nitrate affects groundwater due to the use of nitrogen (N) fertilizers in agriculture and also showed that chemical composition of the water is not only influenced by agricultural practices, but also by interaction with the alluvial sediments [4].

Following up evolution trend of hydrochemical parameters of the “Merdja” aquifer Zereg et al. 2018 confirmed that point sources of nitrate and salinity pollution are particularly downstream in the areas surrounding Tebessa [7].

Multivariate statistical tools in the treatment of analytical and environmental data are very important [8, 9]. Especially Principal Component Analysis (PCA) which is widely used in water quality analysis [10]. The latter assumes a linear relationship between the data and the underlying latent variables represented by the principal components. Because of this we propose to anticipate any non-linear relationship between the data by using Multidimensional Positioning Scaling (MDS), this method assumes no relationship between variables and attempts to optimise the fit between the dissimilarity between observations and their euclidean distance [11].

The purpose of this study is to investigate the hydrogeochemical characteristics of groundwater in the Merdja plain of Tebessa town. Using two multivariate analysis the classical Principal Component Analysis (PCA) and the Multidimensional Positioning Scaling (MDS) method to extract the most weighting variables and also to identify the main causes of the degradation of the quality of these waters.

2. Study Area

The study area totals 288 km² (Figure 1), and extends between latitudes 35°2' and 35°6' North and longitudes 7°87’ and 8°43’ East, upstream of the Mellegue catchment area. The Tébessa plain, with an average altitude of 850m, is drained by the Chabro and El Kébir Wadi, while many narrow and winding ravines, shaped by temporary torrents, drain their waters to the main wadi.

The present zone with a semi-arid climate receives average interannual rainfall not exceeding 350 mm/year, with very significant temperature variations in this region, with the minimum extreme temperature reaching -5°C, while the maximum temperature exceeds 40°C. The rainy season generally extends from winter to spring. The period of low water begins in May and continues until October when there is a high evaporation capacity, unlike infiltration, which represents only 1% of precipitation.

As a result, the supply of water to the groundwater table from rainfall is very low. Due to these climatic conditions, the vegetation is quite poor and sparse. The population is mainly concentrated around the city of Tébessa, whose main activity is agriculture and trade. The industry is relatively under-represented and consists of a few state-owned companies.

Figure 1. Study Area
Along the border of the Maghreb mountain range, the Eastern Saharan Atlas is characterized by subsidence, diapirism, folding, faults and collapse ditches [1, 2].

The Triassic consists of multicoloured gypsum marls, dolomites, dolomitic limestones, sandstones and fragments of green rocks, typical of the Triassic in Algeria. It constitutes Djebel Djebissa (housing an iron deposit) at the far east of the basin, and outcrops in the northwestern part of the basin (Boulhaf-le-Dir). The lower Cretaceous, with a thickness of about 1000 m, is composed of platform carbonates topped with marl, marl and limestone, as well as clay and clay limestone, while the upper Cretaceous, about 2000 m thick, the Campanian and the Maestrichtian are composed mainly of chalky limestone.

The Eocene is composed of 200 m thick flint limestone. The Upper Miocene continental and outcrops south of the Roman Jebel Bou. The Pliocene of marine and continental type. It is composed of conglomerates, marl and sandstone, Quaternary deposits consist of soils with heterogeneous lithology, such as silts, limestone crusts and pebbles.

3. Materials and Methods

3.1. Sampling and Analysis

Water samples (Table 1) were extracted from the wells collecting the groundwater. Samples were collected at the end of the rainy season (May).

The physico-chemical parameters, pH, temperature and electrical conductivity, were measured in situ and the analysis of major ions was carried out by flameatomic absorption for cations and by colorimetry for anions [12] at the laboratory of the National Hydraulic Resources Agency (ANRH) in Constantine.

<table>
<thead>
<tr>
<th>Variable</th>
<th>CE</th>
<th>Ca2+</th>
<th>Mg2+</th>
<th>Na+</th>
<th>K+</th>
<th>Cl</th>
<th>SO4(^2)</th>
<th>HCO3</th>
<th>NO3</th>
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<tbody>
<tr>
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<td>1351</td>
<td>96</td>
<td>58</td>
<td>86</td>
<td>1</td>
<td>110</td>
<td>204</td>
<td>165</td>
<td>1</td>
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<td>66.25</td>
<td>106</td>
<td>1</td>
<td>172.5</td>
<td>351</td>
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<td>84</td>
<td>122</td>
<td>2.5</td>
<td>220</td>
<td>392</td>
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<td>101.5</td>
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<td>409</td>
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<tr>
<td>Max.</td>
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<td>226</td>
<td>124</td>
<td>225</td>
<td>14</td>
<td>375</td>
<td>900</td>
<td>451</td>
<td>120</td>
</tr>
</tbody>
</table>

3.2. Principal Component Analysis

The objective of PCA analysis is to move from data space to factor space in order to reduce the number of variables and make the information less redundant. This is equivalent to replacing the variables \(x_1, \ldots, x_p\) which are initially correlated by new variables called main components \(\xi_1, \ldots, \xi_p\) uncorrelated. These variables are linear combinations of \(x_j\), they are of maximum variance and obtained by inertial maximization.

Principal Component Analysis is widely used for water quality characterization [8, 13-15]. Chaib and Samraoui (2011) applied this method to characterize the waters of the East Kebir Wadi (Algeria), which allowed them to identify the main causes of degradation and to assess the physico-chemical quality of the waters of this and its main tributaries [14].

Mishra (2010), during a monitoring of the Ganges River in Varanasi, used the PCA to extract the parameters that are most important in assessing the quality of these waters and concluded that only four main factors were identified as responsible for the structure of the data explaining 90% of the total variance of the data set [8]. Mahapatra et al. (2012) applied Principal Component Analysis to classify water samples into four different categories, this classification will be useful for planners to take improvement measures in advance, to prevent groundwater contamination [16].

3.3. Multidimensional Scaling (MDS)

Multi-dimensional Scaling [17] or proximity analysis is a special case of multivariate analysis with which one can reconstruct an image in a Euclidean space from one or more tables of distances or dissimilarities between objects (individuals). It aims to best represent objects in a visualizable space, so that the distances between objects in this space are as close as possible to the initial dissimilarities [18]. The optimal space being obtained by minimizing the Stress (Standardized Residual Sum of Squares) criterion [17]. Let \(d_{ij}, \delta_{ij}\) be dissimilarities and real distances, respectively.
\[ \text{Stress} = \left( \frac{\sum_{i,j} (\delta_{ij} - d_{ij})^2}{\sum_{i,j} d_{ij}^2} \right)^{1/2} \]  

(1)

MDS is based on an iterative approach illustrated by the:

1. Initialization (random or not) of the coordinates of the objects in the plane,
2. Calculation of distances between objects in the plane,
3. Calculation of the disparities between the distances in the plane and the distances in the original space,
4. Calculation of a cost function based on these disparities,
5. Update the coordinates of the objects in the plane and return to step 2.

There are two types of MDS: (i) the metric MDS and (ii) the non-metric NMDS the latter has the advantage of having no assumption made about the underlying transformation function [18]. This characteristic allows the technique to be applied to several fields of study [19]. The Non-Metric Multidimensional Scaling (NMMDS) method is based on minimizing the criterion:

\[ \text{Stress} = \left( \frac{\sum_{i,j} f(\delta_{ij} - d_{ij})^2}{\sum_{i,j} d_{ij}^2} \right)^{1/2} \]  

(2)

Where, \( f \) is an increasing monotonous function (to be determined by the resolution algorithm) of dissimilarities [20]. Stress values close to zero indicate that the results of the MDS analysis are reasonable and reliable. Several authors have used MDS to characterize water quality [21, 13].

Wu et al. (2011), using MDS analysis, identified the spatial and temporal variability of hydrochemical water quality in a subtropical coastal system, Daya Bay, China [13]. Group I was identified as being mainly related to anthropogenic activities, both in terms of Group II and in terms of trade in sea water from the South China Sea. Vargas-González et al. [22], based on a NMDS analysis of water samples from the Gulf of California, showed the existence of different hydrological conditions each season. The marked differences between summer and winter are attributed to water temperature and phytoplankton biomass on the one hand and summer conditions in La Salada Cove on the other.

4. Results and Discussion

PCA analysis was performed using R© software with the adopted Factominer, Factoextra and Performance Analytics packages. The link between all the variables taken in pairs and the correlation coefficients between these different variables is given by the correlation matrix (Figure 2). Indeed, perfect correlations were first recorded between Calcium and Electrical Conductivity (r=0.84). This indicates that most of the Electrical Conductivity of the water in the study area comes from Calcium. In addition to being correlated with CE, Calcium and Magnesium are both strongly correlated with each other (r=0.90), which confirms their origin which is the dissolution of carbonate formations, and gypsum formations (evaporite).

Also, a high correlation between Chlorine and Magnesium (r=0.69) was recorded. In addition, sulphates are well correlated with sodium (r=0.6) and a strong correlation with electrical conductivity, and this can be explained by the dissolution of evaporative formations.

However, a high correlation between nitrate and potassium values (r=0.69) was observed, which proves their origin. The dissolution of chemical fertilizers due to fertilizer abuse and a relatively high correlation between Cl and Ca (0.68) also was observed. This is due to the mechanisms for acquiring the salt load of water disturbed by several phenomena, such as the basic exchanges that characterize highly mineralized water, the leaching of evaporation levels and the dissolution of gypsum and halite, which can increase the calcium and chloride ion content respectively.

In addition, a strong relationship between Cl and K (0.74) suggests that some of the chlorides come from the dissolution of KCl but that the majority of these cations could come from the dissolution of other minerals.

The insignificant relationship between sulphates and calcium and magnesium is probably due to the fact that calcium and magnesium ions are involved in dissolution/precipitation phenomena in basic exchanges on clay minerals, whereas TDS is related to the main elements, Ca, Mg, CE and SO4 with a correlation coefficient that varies between 0.67 and 0.97.
To confirm the interpretations made above and to make it easier to visualize the influence of the physico-chemical parameters between them and on the quality of the water of the Tébessa plain. The scree plot has identified two axes that carry 80.3% of the information contained in these variables (Figure 3), while projections made on both axes show that axis 1 summarizes 50.1% of the information and describes the variables related to mineralization (Cl, Mg, SO4, electrical conductivity, Ca, NO3 and K). It can be considered as a gradient of mineralization related to the intensity of pollution provided by urban spills and industrial waste water companies [23].
Projections made on the first two axes (Figure 4) show that: axis 1 summarizes 50.1% of the information and describes the variables related to mineralization: (Cl, Ca, Mg and CE). Axis F2 expresses 30.2% of the initial information, it is positively correlated by nitrates and potassium, it is linked to agricultural activity which is due to the use of fertilizers [4].

![Figure 4. Projection on the first 2 main axes](image)

The MDS analysis carried out with R confirms the results found by the PCA, which has divided the site into three groups of wells, which differ in their physico-chemical quality (Figure 5). The first Group G1 includes wells containing high concentrations of No3, Ca, K, Cl, Cl, Mg, So4 and electrical conductivity which is closely related to pollution caused by urban and industrial wastewater discharges [4, 7].

Concerning the second Group G2, on one hand, it maintains the relationship between the two elements NO3 and K caused by fertilizers used in agriculture [4, 7] and on the other hand, the relationship between Ca, Mg and SO4 due to the geological formations of the region with average values [6, 7]. Group3 reflects the strong relationship between TDS, Mg, SO4 and Ca which reflects the salinity of this groundwater resulting from the long-term interaction between it and the different geological formations [7].

![Figure 5. MDS Clustering](image)
5. Conclusion

The study carried out in the Tebessa plain highlighted the main characteristics of groundwater. The groundwater has high concentrations of nitrates, which can reach 120 mg/L. Contamination results mainly from the spreading of fertilizers and waste that is discharged into the plain without treatment. However, a strong correlation between nitrates and potassium proves that they are of anthropogenic origin.

The strong relationship between TDS and Ca, Mg, CE and SO4 indicates that these elements contribute to groundwater salinity and that groundwater follows a similar pattern. Groundwater salinization is a result of long-term interactions effects between groundwater and geological formations. On the other hand, the insignificant relationship between sulphates and calcium and magnesium is probably due to the fact that calcium and magnesium ions are involved in dissolution/precipitation phenomena in the basic exchanges on clay minerals.

6. Conflicts of Interest

The authors declare no conflict of interest.

7. References


A Comprehensive Numerical Study on Building-Excavation Interaction

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Abstract

This paper presents results of a plane strain comprehensive numerical study on the interaction between a 31-meter-deep excavation and an adjacent 12-story building; the study emphasizes on parametric analyses with respect to the building characteristics, such as the building width in plan (B), i.e., the side perpendicular to the excavation wall, the embedment depth of the building foundation (D), as well as the building distance to the excavation edge (e). Through the parametric analyses and assuming different values for B, D, and e, settlements and rotations of the building and horizontal displacements of the excavation edge were computed and evaluated using the finite element method adopted in PLAXIS 3D software. Prior to the parametric study, the numerical modeling was verified by modeling a recorded case study, which is an anchored deep excavation adjacent to a 12-story building. The results of the parametric analyses suggest that for the given soil and excavation, (1) the position of the developing potential failure surface, PFS, in the soil behind the excavation is almost independent from the building location and (2) the position of the building with respect to the outcrop of the PFS in the excavation crest, i.e., if the building locates fully on the potential failure wedge or PFS intersect the building base, is the main factor affecting the induced displacements and rotations of the building.

Keywords: Deep Excavation; Adjacent Building; Failure Wedge; Numerical Analysis; Parametric Study.

1. Introduction

Rapid urbanization has led to growing number of activities, including construction of underground spaces that require cutting of deep excavations proximate to existing buildings. Excavations inevitably induce significant changes in stress/strain states of the surrounding soil, which in turn may cause different levels of displacement and damage in nearby structures [1]. These effects are related to a host of parameters, namely, soil properties, specifications of excavation support system, building properties and location with respect to the excavation, and excavation sequences and geometry [2-7]. In design and construction of deep excavations, it is prudent to avoid excessive lateral displacements and ground surface settlements, by carefully considering the effects of the above-mentioned factors, otherwise uncertainties would often force engineers towards inefficient constructions with high costs [8].

Previous studies have demonstrated that buildings weight, stiffness and position relative to excavations are the main building-related factors influencing excavation-induced deformations of the nearby buildings [3, 9-10]. The stiffness of an adjacent building affects excavation-induced settlements. The bending stiffness and shear stiffness of a building mostly depend on stiffness of frames and in-frame walls, respectively. There are two main methods to include effects of adjacent buildings in numerical modeling of excavations: (1) using an equivalent beam and (2) modeling the building as a frame. In the former, estimation of the stiffness would be appropriate for a building with wall behavior dominancy.

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Despite its widespread use, alternating a simple beam instead of a framed building to represent its real behavior is under discussion [11-13]. Using equivalent surface beam in numerical studies about building-tunneling interaction overestimates stiffness of the building and, therefore, underestimates the induced settlements [14, 15].

Having used the equivalent beam method, Goh and Mair [10] presented building modification factors applicable for determination of adjacent excavation effects on the building deflections and strains. They employed 2D numerical models of weightless framed buildings, with both mat and spread footings, on the crest of a strutted excavation in a soft clay. They also used an equivalent simple beam to represent bending stiffness of the building frame, while neither weight of the building nor the position of the building in relation to the excavation’s Potential Failure Surface (PFS) were taken into account [10].

Son and Cording [16] modeled the building and showed that openings (i.e., doors and windows of masonry buildings) reduce significantly the building’s shear stiffness (compared to the bending stiffness). They suggested that 30% increase in walls openings ratio results in 45-61% reduction in shear stiffness of the equivalent wall [16].

Other studies have indicated that factors such as building structural system, cracking in frame or load-bearing walls and soil conditions affect the response of framed structures to settlements induced by adjacent excavations [17-19].

Thus far, the two main approaches implemented in estimating excavation-induced vertical and horizontal displacements can be classified as empirical (or semi-empirical) methods and numerical modeling. The complex soil-structure interaction can be taken into account only using numerical methods along with an appropriate constitutive law for the soil behavior. Nevertheless, it should be noted that neither of the two methods could be implemented alone if deformations occur due to improper soil excavation or local water leakage into the excavation [20]. However, it is expected that including actual conditions encountered during excavation in numerical models can yield reasonable results.

Application of free-field settlements of excavations to their adjacent buildings has been commonly adopted as a shortcut method for estimating levels of damages to the buildings; of course, this approach ignores effects of the building on the settlement and thus is not adequate for all circumstances [5]. Son and Cording [9] numerically investigated responses of buildings with different structural types, founded on soils with different properties, subjected to progressive free field ground settlements obtained from typical field observations. They indicated that the structural response to excavation-induced ground settlements is highly dependent on the structural type and soil conditions.

Moreover, in order to account for building-excavation interactions in conditions in which the side corners of the excavation are not far enough from the building and also for special geometries of narrow, deep excavations, only 3D models should be used [21-23]. Meanwhile, attempts have been made to correlate plane-strain analyses results to their corresponding 3D analyses [24-25].

Owing to the variety of parameters involved in the interaction behavior of excavations and nearby buildings, no comprehensive study considering the impact of all influencing parameters has so far been conducted. The deformations seem to be better interpreted by considering the building position with respect to the Potential Failure Wedge (PFW) developed in the soil during excavation; this has not been accounted for in the literature. Moreover, stability analyses regarding the building location in relation to the excavation wall have been rarely taken into account. The reason may be attributed to the difficulties involved in investigating simultaneously the excavation stability and induced deformations in a parametric study.

This study attempts to understand how specifications of a framed building in the vicinity of a deep excavation affect the excavation induced deformations. To this end, the PLAXIS 3D-2017 software is incorporated to perform Finite Element analyses. The modeling approach is verified first using the monitoring data from a 31.2 m deep excavation and then used for numerical modeling of a series of anchored piled excavations. Extensive sensitivity analyses with regard to the parameters related to the building’s location (range of which were determined based on the results of the stability analyses) are carried out. The results of the analyses are presented in terms of rotations and settlements of the building and displacements of the excavation crest.

The research methodology of the present study is presented in a flowchart as depicted in Figure 1.
Figure 1. Flowchart of the research methodology (PFS: potential failure surface, main variables: B (building width perpendicular to the excavation wall), e (building distance to the excavation edge) and D (embedding depth of the building foundation), SF: safety factor, θ: building rotation, Uc: excavation edge horizontal deformation, Sb: maximum building settlement)

2. Case Study

The case under study herein is a part of the west side of a 31.2 m deep excavation project, adjacent to a 12-storey building in north of Tehran. The retaining system of this part of the excavation is anchored piles together with a layer of reinforced shotcrete. Geotechnical investigations in the site of the project characterized the soil as a relatively compact, granular, cementitious soil. The ground water table was lower than the excavation base level. Figure 2 depicts plan view of the excavation site and two photos from the excavation wall and the building.

2.1. Support System

The support system applied to provide stability and control deformations in vicinity of the building utilizes the combination of pile, anchorage, wire mesh and shotcrete. After drilling the pile locations up to the depth of 34.2 m
(embedded in concrete with depth of 3 m, below the excavation bottom to provide fixity), steel piles (2IPE360) with 3 m spacing were installed. In each sequence of soil excavation, one row of post-tensioned anchors together with a layer of reinforced shotcrete were implemented. Thickness of the shotcrete layer was about 10 cm. Two sets of four and six multi strand anchors (0.6 inch diameter for each strand) with 150 kN post tension force for each strand were used (Figure 3). Therefore, post tension force for each anchor in the upper three rows and in the lower seven rows of anchors were about 600 kN and 900 kN, respectively.

Figure 2. Excavation Plan of the Case Study and View of the building and the excavation wall

Figure 3. Specifications of the excavation support system
2.2. Building Characteristics

A 12-storey square-shaped (in plan) concrete building, including two basements, with the width of 30 m and a 1.5 m thick mat foundation is located in the west side of the excavation (Figure 2). The horizontal distance between the building and the excavation wall (e) is 5 m and the embedment depth of the foundation is 4.5 m. Regarding the two basements, retaining walls with a thickness of 20 cm and floors of 20 cm thick two-way concrete slab have been constructed. Spacing of the columns (bay widths) is almost 6 m and shear as well as partition walls have occupied around 60% of the frames. In average, the columns are 70 cm in dimension while the beams are 30 cm wide and 50 cm deep.

2.3. Instrumentation and Monitoring

Different instruments such as extensometers, load-cells and tilt-meters were employed to monitor the deflections, anchors forces and building rotations respectively. However, the instruments’ data were not trustworthy due to their low quality and improper/incorrect installation. Therefore, the measured displacements of surveying target points (three on the building, B-U-1, B-M-1 and B-D-1 and one on the excavation wall, W-1-2) as shown in Figure 4 were used for verification of the numerical modeling.

3. Numerical Modeling

Numerical analyses of the interaction behavior include modeling of both the building and the excavation in four modes of Soil, Structure, Mesh and Staged construction. The numerical modeling and analyses employed the elasto-plastic Hardening Soil (HS) constitutive model adopted in PLAXIS 3D [21]. The HS model differentiates between the loading moduli (i.e., $E_{50}$ or $E_i$ in Figure 5a) and the unloading/reloading ($E_{ur}$) modulus, and can be confidently used for deep excavation analysis [26]. This model is based on the Mohr-Coulomb strength criterion and two families of yielding surfaces. The yield surfaces are the “yield cap surface”, with an associated flow rule, taking into account volumetric plastic strains and the “shearing yield surface” which is used to compute distortional plastic strains. The flow rule is non-associated and the plastic potential function is defined to ensure a hyperbolic response [27]. The stress-strain behavior of the HS Model is depicted in Figure 5 and the Model parameters and their assigned values for the numerical modeling of the case study as well as the fore-coming parametric study are presented in Table 1. By defining a borehole in the Soil mode of the modeling process, the thickness and constitutive model of the soil layer were assigned.

![Figure 4. Selected target points on the excavation wall and the adjacent building](image)

![Figure 5. Characteristics of Hardening Soil model](image)
Table 1. Soil Properties and HS Model Parameters used in numerical analyses

<table>
<thead>
<tr>
<th>Property</th>
<th>$E_{50}^{\text{ref}}$</th>
<th>$E_{0}^{\text{ref}}$</th>
<th>$E_{ur}^{\text{ref}}$</th>
<th>Power, m</th>
<th>$c'_{\text{ref}}$</th>
<th>$\Psi$</th>
<th>$\psi'_{ur}$</th>
<th>$P_{\text{ref}}$</th>
<th>$K_0$</th>
<th>$\gamma$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case study</td>
<td>60</td>
<td>60</td>
<td>300</td>
<td>0.6</td>
<td>50</td>
<td>39</td>
<td>5</td>
<td>0.2</td>
<td>100</td>
<td>0.37</td>
</tr>
<tr>
<td>Parametric study</td>
<td>60</td>
<td>60</td>
<td>300</td>
<td>0.5</td>
<td>25</td>
<td>36</td>
<td>0</td>
<td>0.2</td>
<td>100</td>
<td>0.41</td>
</tr>
</tbody>
</table>

* "ref" denotes the reference pressure for triaxial or oedometer test in which the model’s parameters should be determined.

The structural elements of the building (i.e., beams, columns, slabs, in-frame walls and foundation) as well as the support system (i.e., piles, anchors and reinforced shotcrete layers) were created in the Structure mode in PLAXIS. The structural model studied here is a 6 m wide slice from the 30×30 m square shaped in-plan building. As shown in Figure 2, the building is about 15 m away from the excavation corner; therefore, plane strain conditions with an acceptable accuracy applies to the analysis. To ensure a representative frame for the building (regarding the typical 6 m bay width of the building frame), two anchored piles were incorporated into the model. The unit weight of the structural elements of the building were taken into account by assigning 12 kPa equivalent load (sum of the dead load plus the live load) for each floor of the building.

The shotcrete and piles were modeled with plate elements, while for the bond length of the anchors, embedded beam elements were used. It is worthwhile mentioning that plate elements for modeling the piles were selected to account for the lateral earth pressure on them [27]. Moreover, a block of "volume" element with concrete properties was added to the plate element to model the embedded part of each pile. Furthermore, structural elements such as beam, column, slab, foundation and retaining and in-frame walls were modeled as plate elements.

The PLAXIS 3D program allows for a fully automatic generation of finite element meshes which takes into account the soil layer as well as all structural elements and boundary conditions. After defining the structural elements, mesh generation process in the Mesh mode, by selecting finer mesh sizes around the building zone of the soil body, was conducted. The generated mesh and model’s geometry are shown in Figure 6a.

The finite element calculations involve several sequential phases. Each calculation phase corresponds to a particular loading or construction stage. After initial $K_0$ stress analyses in the first phase of the Staged construction mode, the adjacent building and the piles were erected in the second phase of the modeling. Then in the third phase, the first stage of excavation and activation of the supporting system (i.e., anchorage and mesh installation and shotcreting) was modeled according to the practice and excavation geometry of the real project. This phase was repeated for 9 more stages until reaching the final excavation level. Deformations computed at the end of each phase were computed and recorded; in reality however, only deformations pertinent to the excavation stages are of concern.

3.1. Verification of Numerical Model by the Case Study

The soil removal in vicinity of the building was carried out in the sequence as depicted in Figure 6b. The actual excavation sequence was obtained using the available photos/reports. There is a real doubt regarding the efficiency of this excavation sequence as far as controlling of ground deformations [28] is concerned. However, the same excavation sequence was employed in the numerical modeling to follow the real field conditions. Simultaneous with each layer of soil removal, a row of anchors was activated.

Figures 7 shows the comparison of the measured and computed horizontal displacements and settlements of the building and excavation at the target points, during excavation. As seen, the computed results and surveying data are in good harmony with each other in all of excavation stages; this verifies the numerical modeling approach.

![Figure 6](image_url)

(a) Mesh idealization

(b) Real excavation stages of the case study
Further sensitivity analyses showed that the most favorable results are obtained when the bottom boundary condition is about 12.5 m below the excavation floor. Accordingly, in the cases where the soil layer at the bottom of the excavation are thicker (i.e., more than 12.5 m in this model), the computed horizontal and vertical displacements differ substantially from the measured ones due to unreal heave of the soil layer beneath the excavation bottom.

### 3.2. Parametric Study

The stiffness and weight of the adjacent building are the two most building-related factors influencing excavation-induced deformations according to the literature. Another important factor affecting the behaviors of both building and excavation is the relative position of the building with respect to the Potential Failure Surface (PFS), which develops within the soil. This relative position depends, of course, on the embedment depth (D), width (B) and distance of the building from the excavation (e), as shown in Figure 8.

Considering all of the above factors and selecting the soil, excavation, and building (i.e., 12-story and 36 m tall) similar to those of the case study introduced in Section 3, totally 36 building-excavation models were developed. Table 1 presents the parameters of the soil and the 31.2 m deep excavation wall.

Deformations of the excavation and the building depend on the excavation support design, which in turn is a function of soil characteristics as well as the loading (building location, width and depth). Two approaches may be opted for in the parametric study of the excavation-building interaction. The first approach is to assume an identical Support of Excavation (SOE) for different loadings, and the other one is to consider an identical Factor of Safety (FOS) for the SOEs under different loadings; obviously in the latter approach, the SOE characteristics are case-dependent and depend on the loading conditions. The first approach was adopted in this study. For a realistic comparison of the results, however, the ranges of the parameters of interest were defined in a way that the stability of the SOE is satisfied, in an allowable range of factor of safety, for every loading conditions assumed. To this end, the admissible range of the variables given in Table 2 were assigned based on the stability analyses carried out by Geoslope software, with allowable FOS ranging between 1.2 to 1.6.
Figure 8. Problem Description; (a) main variables in the parametric study (the building horizontal distance from the excavation \(e\), foundation depth of the building \(D\) and the building width \(B\)), (b) response parameters (horizontal displacement of crest \(U_c\) and maximum settlement \(S_b\) and rotation \(\theta\) of building).

Table 2. Values of the main variables applied in parametric study

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Assumed values (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>12, 18, 30</td>
</tr>
<tr>
<td>E</td>
<td>0, 6, 12, 18</td>
</tr>
<tr>
<td>D</td>
<td>2, 4.5, 6</td>
</tr>
</tbody>
</table>

Table 3. The cases with the most and the least safety factors in the parametric study

<table>
<thead>
<tr>
<th>Model</th>
<th>(e) (m)</th>
<th>(D) (m)</th>
<th>(B) (m)</th>
<th>F.S.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case study</td>
<td>6</td>
<td>4.5</td>
<td>30</td>
<td>1.4</td>
</tr>
<tr>
<td>Most stable</td>
<td>18</td>
<td>6</td>
<td>12</td>
<td>1.6</td>
</tr>
<tr>
<td>Least stable</td>
<td>0</td>
<td>2</td>
<td>30</td>
<td>1.2</td>
</tr>
</tbody>
</table>

The results are presented in forms of horizontal displacement of the excavation crest \(U_c\), maximum settlement of the adjacent building \(S_b\) and rotation of the building \(\theta\), as shown in Figure 8b; these parameters are interpreted based on the concept of relative position of the building with respect to the PFS. For a better interpretation of the results, a simplified statement has been developed in a conceptual framework related to free field conditions (see Sec.4.2).

4. Results of Parametric Studies

PFS positions within the soil for different values of the main variables is defined in this section and thus the results of the parametric study are interpreted on the grounds of the building's position in relation to the PFS position along with the contribution of the building stiffness and weight in the failure wedge.

4.1. Potential Failure Surface Position

For investigating the dependency of the response parameters to the situation of the building relative to the PFS, it is needed to determine PFS position in the models. The location of PFS can be determined by means of the mobilized shear stress ratio, \(\tau_{rel}\) (Figure 9a), total principal strain value and direction in the soil (Figure 9b), and the deformed finite element mesh (Figure 9c); however, the recent approach is not as precise as the two former methods.

Figure 9. Three methods for assessment of the failure wedge in finite element models

Inspection of relative shear stresses and principal strains directions in Figure 10 (e.g., \(e=3\) m) reveals a clearly distinguishable shear band, delimited in red color. Moving away the building (e.g., \(e=12\) m in Figure 10) results in fading
of the red band of the PFS and appearance of local red zones (with high relative shear stress) at immediately below the foundation corners. However, the trace of the red band as an indication of PFS still exists in the lower elevations of the soil near the excavation toe. It is noteworthy that the local effects of strain localization at the corner of the building foundation is ignored and the location of the PFS is mainly determined on the basis of shear band trace passing through the excavation toe.

Figure 10 illustrates changes of PFS position in the soil body for different values of e. As seen, changing the building distance from the excavation wall (e) has negligible effects on the PFS position. It can be realized that changing the building width (B), does neither distinguishably alter the PFS position (Figure 11). In other words, the PFS resists against changes in its position for different values of e and B of the building. Similarly comparing the PFS position for different values of the foundation depth of the building indicated that increase in D values also has no noticeable effect on the PFS position. Thus, it can be concluded that the PFS position is almost independent of the main parameters (i.e., B, D and e) considered in the analyses.
Figure 10. Indication of potential failure surface for different values of $e$ ($B=30$ m for all cases)

<table>
<thead>
<tr>
<th>$B$ = 12 m</th>
<th>$B$ = 18 m</th>
<th>$B$ = 24 m</th>
<th>$B$ = 30 m</th>
</tr>
</thead>
<tbody>
<tr>
<td>(a)</td>
<td>(d)</td>
<td>(g)</td>
<td>(j)</td>
</tr>
<tr>
<td>(b)</td>
<td>(e)</td>
<td>(h)</td>
<td></td>
</tr>
<tr>
<td>(c)</td>
<td>(f)</td>
<td>(i)</td>
<td></td>
</tr>
</tbody>
</table>
4.2. Building Rotation (θ)

This section investigates variations of building rotation (θ) because of excavation-induced deformations. Developing a conceptual framework, in the first stage, the impacts of the location of a hypothetical building with respect to the PFS (Figure 12a) on the building rotation are described based on the ground surface settlement profile in the free field model (Figure 12b); θ is calculated based on differential settlements of the mat foundation. Figure 12c illustrates θ variations of a hypothetical building with D=0 versus "e+B/2" for different values of B and e. Here, "e+B/2" is the distance of the building foundation midpoint from the excavation edge. Comparison of the building rotation for different values of e and B in Figure 12c implies that the rotation direction of the hypothetical building alters almost at e+B/2=20m, corresponding to the point of maximum ground settlement in free field conditions (Figure 12b).

The location of the maximum settlement may correlate geometrically to the PFS location. In other words, for e+B/2<20m the building mainly locates inside the PFW (where rotations are counter clockwise according to the ground surface settlement profile). As e+B/2 exceeds 20 m the building tends to move outside the PFW, with smaller settlements and yielding clockwise rotations. Thus, the rotation of the hypothetical building may correlate to its relative location with respect to PFS.

This trend is also obvious for real buildings as were indicated in Figures 10 and 11, in which vertical black lines under the buildings (in the figures related to total principal strain directions) show the location of building midpoint with respect to PFS. It is of interest to compare the rotation direction of the corresponding building in the deformed mesh figures with the position of the above vertical black line in Figures 10 and 11. It can be seen that in the presence of the real building, zero rotation also occurs when e+B/2≈20m. This compatibility suggests that the building rotation is highly dependent to its relative position with respect to the PFS.
Figure 12. (a) Potential failure surface position in free field condition, (b) ground surface settlement profile together with slip surface position in free field condition and (c) the hypothetical buildings’ rotation for different values of “e+B/2” in free field condition

Settlement profiles versus e for different D values of hypothetical buildings in free field conditions are depicted in Figure 13a. As D values increase, the corresponding soil volume and weight are removed and settlements decrease due to the reduction in the failure wedge surcharge. The decrease in settlements leads to the reduction of differential settlements and subsequently the rotations.

Figure 13b depicts variations of θ versus e+B/2 for different values of D in free field conditions. The trend lines, which have been fitted, represent the ground surface slope. As seen, higher values of D result in lower rotations, both clockwise and counter clockwise, of the building.

Figure 13. (a) Settlement profiles for different D values of hypothetical buildings in free field condition, (b) The hypothetical buildings’ rotation for different values of D in free field condition
In addition, by increasing embedment depth of the building (D), the part of the building which falls outside of the failure wedge becomes relatively larger; compare together the positions of the building in depth D1 and D2 in Figure 14. It is obvious that as the D becomes larger the chance for clockwise rotation of the building increases.

Figure 14. Effects of D on a building position with respect to the PFS

Given the above conceptual framework for the building-excavation interaction, which developed based on the free field model, a series of parametric studies were carried out for different values of D, B, and e. Figures 15a, 15b, and 15c illustrate the building rotation (θ) versus B and different e values for D=2 m, 4.5 m, and 6 m, respectively. According to these figures for given D and e, θ increases generally (i.e., switch to more a clockwise direction) as B becomes larger. This was explained in Sec 4.1 (Figure 14) that the building moves toward the PFS boundary as B becomes larger. There are a few exceptions (e.g., e=0 and 6 in Figure 15a and e=0 in Figure 15b) in which increasing of B from 12 m to 18 m has rotated the building in counter clockwise direction. This is because in these cases, the building is still inside the PFS with a safe margin; however, the weight of the building (due to enlarging B) has increased, and lead to rotation that is more negative. This exception is suppressed as D increases in Figures 15c.
Figure 15. Building rotation (θ) for different values of B, D and e in the parametric study

Figure 15d, 15e, and 15f present the same results of the parametric study in terms of the building rotation (θ) versus e+B/2 and different B values for D=2 m, 4.5 m, and 6 m, respectively. As seen, the results, including the trend lines, follow the same general trend, and they are favorably comparable with the conceptual framework introduced in Figure 12c.

Moreover, the trend lines of θ variations from the above three figures are redrawn in Figure 15g, which compares favorably with the conceptual framework for the free field model shown in Figure 13b.

4.3. Excavation Wall Horizontal Displacement (Uc)

We expect that Uc be dependent upon B, D, e and more importantly building location with respect to the PFS. Plots of Figure 16 assess the level of dependency of Uc to each of the above variables. In this regard, Figures 16a, 16b and 16c reveal two important findings, as follows. (1) Weight of the building (due to increasing of B while most part of the building still inside the PFW) is responsible for the cases in which an increase in B is followed by an increase in Uc. (2) Locating most part of the building (due to extension of the building width, B, beyond the PFS boundary) is responsible for the cases in which an increase in B has led to a decrease in Uc. In other words in the latter, that part of B, and its corresponding weight and stiffness, beyond the PFW has controlled and restricted horizontal movements of the excavation.

Figures 16d, 16e, and 16f suggest that for a given B, horizontal displacement of the excavation crest, Uc, proportionally decreases with D for all of the e values. The observed Uc reductions by increase in D, is attributable to two main reasons; i) the soil weight removal corresponding to the lowered depths of the building foundation which cause surcharge reduction on the wedge, and ii) the fact that more parts of the building exit the PFW area due to smaller failure wedge. It is instructive that for B=12m, in which B+e is less than the width of PFW, the variation of Uc is almost independent of e.
Considering what we elaborated in Section 4.1 regarding the potential failure surface, PFS (Figures 10 and 11) and Section 4.2 (building rotation, \( \theta \); Figure 12), one expect that like \( U_c \), the maximum settlement of the building (\( S_b \)) be dependent upon variables such as \( B \), \( D \), \( e \), weight of the building, and its location with respect to the PFS. Another parameter that may affect \( S_b \) is the proximity of the building and particularly the location of its maximum settlement to the supporting system of the excavation wall, as indicated in Figure 13a; the supporting system generally enhances the global stiffness of the soil that limit the soil settlements in this area. Having all of the foregoing explanations in mind, Figure 17 shows variations and dependency of \( S_b \) with regard to the varying parameters of \( D \), \( B \), and \( e \).

The values and trends of these variations can be explained and justified one by one, considering specific effects of each influencing parameter. For the purpose of brevity, this explanation is not presented here.

**4.4. Maximum Settlement of Building (\( S_b \))**

Figure 16. Horizontal displacement of the crest (\( U_c \)) for different values of \( B \), \( D \) and \( e \) of building
Figure 17. Maximum building settlement ($S_b$) for different values of B, D and e
5. Conclusions

This paper presented results of a parametric numerical study to investigate building related parameters (i.e., building width, B, embedment depth, D, and building distance from the excavation crest, e) affecting the interaction of an anchored piled deep excavation in a relatively dense granular cementitious soil with a nearby framed building. The numerical modeling approach in the parametric study was verified by a case study. Horizontal displacement of the excavation crest ($U_c$), maximum settlement of the adjacent building ($S_b$) and rotation of the building ($\theta$) were investigated in the parametric study. The main results drawn from this study are summarized, as follows:

- Building characteristics have little impact on the position of the potential failure surface, PFS, in the soil developed by the excavation.
- The building position in relation to the potential slip surface is an important factor affecting rotations and displacements of the building due to excavation. This factor provides a useful framework for interpretation and prediction of the responses ($\theta$ and $S_b$) of the nearby building, as well as the excavation wall, $U_c$.
- The rotation of the building is interpreted based on the free field model. As long as the building locates inside the potential failure wedge, the rotation is counter clockwise (for the geometry of this research). As the building moves away from the excavation, its rotation reduces gradually and when the building distances more from the excavation edge (i.e., some part of the building locates outside of the potential failure wedge, PFW), the building rotation shifts to the clockwise direction.
- Increasing the building embedment depth, D, reduces width of the failure wedge at the level of foundation base. In other words, increasing D has the same effects as of moving the building outward the failure wedge. This means that for a given B, increasing D pushes the building toward outside of the PFW.
- The building’s position relative to the potential failure surface together with the effective surcharge on the wedge (weight of that part of the building inside PFW) and the building stiffness are responsible for the rotation, $\theta$, and settlement of the building, $S_b$, as well as the horizontal displacement of the excavation crest, $U_c$.

6. Acknowledgements

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7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


Research on Application of Buckling Restrained Braces in Strengthening of Concrete Frame Structures

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Abstract

This paper examines the application of BRB in strengthening of reinforced concrete frame structures to meet seismic requirements according to Chinese seismic design code. Elastic response spectrum analysis and nonlinear time history analysis are performed by taking a real engineering example that suffers weak first floor irregularity due to added loads and addition of one floor. With the method of equivalent stiffness and displacement-based design method, buckling restrained brace parameters are deduced and are used to model BRB in ETABS using plastic wen model. Three configurations of buckling restrained braces are studied together with ordinary braces. Under elastic state, the relationship between the required cross section area of BRB and ordinary braces is deduced from the formula of calculating elastic bearing capacity where it is shown that the area of the ordinary braces must be 1.25 times that of BRB for ensuring the same performance. The results show that Inverted V brace configuration demonstrated better performance over single brace and V brace configurations and X brace configuration, although not recommended by Chinese code, is simulated and used in this paper and has demonstrated better performance over other configurations, and the further research on the practical use of this brace is recommended. Also, under action of strong earthquakes, by nonlinear time history analysis, buckling restrained braces demonstrated better performance of strengthening the structure and make it meet the requirement of code. Under this same condition, ordinary braces losses their bearing capacity due to excessive buckling.

Keywords: Buckling Restrained Brace; Flexible First Story; Response Spectrum; Nonlinear Time History Analysis; RC Frame Structure.

1. Introduction

Located between the Eurasian seismic belt and the circum-Pacific seismic belt, China is one of the countries with the largest number of earthquakes in the world and one of the countries with the greatest loss due to earthquake disasters because of its complex geological structure and frequent seismicity [1]. And with the steady development of Chinese cities vertically and horizontally, building codes are regularly revised to make sure that the safety is assured and meet the current status.

As the building codes become updated, some of the structures that have been designed by referring to the past codes do not meet the current standards and need to be rechecked and reinforced where needed. May researches have been done for mitigating earthquake disasters through improving seismic performance of buildings by incorporating energy dissipation dampers [2]. This was done to improve the seismic performance of moment frame structures that do not meet seismic requirements under sudden earthquakes.

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Central braces that have been a preferable method in increasing the lateral stiffness of the moment frames have low ability in energy dissipation due to buckling under compression and this make central braces not be able to achieve desired results [3].

In 1994, after Beijing earthquakes, engineers started to become hesitant in using central braces due to the failure of many building frames that were built by central braces. The research on ordinary central braces have shown that they can significantly improve the bearing capacity and lateral stiffness of frame structures but they cannot maintain the safety of frames under strong earthquakes, also because design of ordinary central frames is controlled by stability of compression bars, their slenderness ratio is very limited and this leads to large cross-section[4]. The big cross section increases the weight thus the seismic response of the structure is increased correspondingly and it can also affect the bearing capacity of the structure.

The research on the application of BRB in frame structures have shown that buckling restrained braces yield ductility in both compression and tension, they characterized by a full, stable and symmetric hysteretic loop with relatively low yield stiffness [5]. Although these two kinds of braces have been studied intensively in design of frame structures, their application in strengthening of reinforced frame structures against lateral and vertical loads is still not fully studied. There are many structures’ circumstances that have not yet studied as far as using BRB in strengthening of reinforced frame structures. One of these circumstances is first weak story case that is used in this paper.

Taking the Flue Gas Desulfurization System engineering of Manzhou (2X200MW) Thermal Power Project building of Manzhou city in China as engineering case, based on ETABS finite element software, the response spectrum analysis according to Chinese code of seismic analysis of building structures and the nonlinear time history analysis under multiple earthquakes are carried out to understand the seismic performance of reinforced concrete frame structure strengthened by buckling restrained braces. In this paper, also the two the difference between buckling restrained braces and ordinary braces is highlighted.

It is common in engineering that some additional floor or loads that were not considered during analysis and design can be added to the buildings after construction. That additional load on the building increases mass of the super structure and this results in increase of seismic reaction of the structure and the vertical load, which will also affect the vertical load bearing system of the structure. Also, the seismic capacity of the structure that have been built before updating the codes cannot be considered to be safe without checking it against updated codes. It is very important to strengthen the existing building structures that do not conform to the current codes because significant human and economic losses have occurred in earthquakes [6, 7] and many countries cannot afford to demolish and build new structures

Buckling restrained braces have been proven efficient in increasing the seismic stiffness of the structure because of their full hysteretic curve [8, 9]. Results from the experiments conducted by Mazzolani et al. [10], on retrofitting of damaged structures showed that the seismic performance of steel structures can be improved by both buckling restrained(BRB) braces and eccentric braces, but the first provided larger displacement capacity and they proved that with BRB the increase in stiffness and strength can be better controlled. Kassai et al. [11], by using shaking table, conducted an experiment on a full scale five story steel frame structure with four different energy dissipation devices where BRB was among them; the frame structure was subjected to low intensity white noise and Takatori motion of the 1995 Kobe earthquake scaled to different intensity levels and the seismic performance results of the structures without and with these devices installed on four of five stories were compared where reduction in displacements, floor accelerations and story shears were reported on structures with BRB.

Hector et al[12] studied the effect of BRB on reinforced concrete precast structures, using shaking table experiments and the results showed that inclusion of BRB increased shear capacity, damping and fundamental natural frequency and reduced interstory drift and lateral displacement when subjected to low intensity and high intensity ground motions.

Significant increase in strength, deformation and energy dissipation capacity of reinforced concrete building structures due to inclusion of BRB was also confirmed by Andre Almeida et al. [13] in their research conducted by taking pre-code building structure. These are seismic characteristics that are important in limiting damage in the original structural elements.

The following gaps have been found from literatures:

**First.** most of the researchers have been focused on the application of BRB in design of new buildings. Therefore, there is a need to strengthen the research on the application of BRB in strengthening of existing structures.

**Second.** while in China, most of the buildings are reinforced concrete, researches on BRB are mostly applied in steel structures, therefore is beneficial to do research on the reinforced concrete structures as well.

**Third.** being very new, the research on the effect buckling restrained brace on the vertical and horizontal irregularity of frame structures has not been exhausted. This paper investigates the use of Buckling Restrained braces in strengthening flexible first story building due to added machine load and one floor that was not considered during design and construction stage.
The methodology used in this paper is summarized in the Figure 1:

![Figure 1. Flowchart of the research methodology](image)

2. Theoretical Analysis of Buckling Restrained Braces

2.1. Equivalent Stiffness of Buckling Restrained Braces

Elastic Stiffness

Buckling restrained braces is an assembly of three components (Figure 2). Those are yielding steel bar, encasing material and restraining tube. But also, longitudinally, it has three sections: working section, transition section and connection section.

![Figure 2. A typical Buckling restrained brace](image)

By principles of mechanics of materials, the stiffness of each component can be obtained and the equivalent stiffness can be deduced. Assuming that the brace remains in elastic range;

Displacement of working section is calculated by Equation 1 as follows:

$$\Delta l_1 = \int_{l_1}^{pdx} \frac{pdx}{EA_1} = \frac{p_{l_1}}{EA_1}$$

And from Hooke’s law, the corresponding elastic stiffness is calculated from Equation 2:

$$K_1 = \frac{p}{\Delta l_2} = \frac{EA_1}{l_1}$$

For transition section, displacement is calculated by Equation 3:

$$\Delta l_2 = \int_{EA_2}^{pdx} \frac{pdx}{EA_2} = \frac{p_{l_2}}{EA_2} \left( \ln A_1 - \ln A_3 \right)$$
And the elastic stiffness can be calculated by Equation 4:

\[
K_2 = \frac{P}{\Delta l_2} = \frac{E(A_1-A_3)}{I_2(\ln A_1-\ln A_3)}
\]

Or:

\[
K_2 = \frac{E A_2}{l_2}
\] (4)

And the connection section, displacement is calculated by Equation 5:

\[
\Delta l_3 = \int \frac{Pdx}{EA_3} = g \frac{P l_3}{E A_3}
\]

And the corresponding stiffness is deduced by Equation 6:

\[
K_3 = \frac{P}{\Delta l_3} = \frac{E A_3}{l_3}
\] (6)

The equivalent stiffness of Buckling restrained brace can be calculated by the principle of stiffness of elements in series by Equation 7:

\[
K_e = \frac{1}{K_1 + \frac{1}{K_2} + \frac{1}{K_3} + \frac{l_1 A_1 A_3 + 2 l_2 A_1 A_3 + 2 l_3 A_1 A_3}{E A_1 A_2 A_3}}
\] (7)

Where; \( P \) is the axial load on BRB core member; \( E \) is the elastic stiffness; \( l_1, l_2, l_3 \) length of working section, transition section and connection section respectively; \( \Delta l_1, \Delta l_2, \Delta l_3 \) axial deformation of working section, transition section and connection section; \( A_1, A_2, A_3 \) Cross sectional area of working section, transition section and connection section; \( K_1, K_2, K_3 \) Elastic stiffness of working section, transition section and connection section; \( K_e \) Equivalent stiffness.

Plastic stiffness

For better energy dissipation effect, in nonlinear stage, the working section will be yielded before transition and connection section [14]. This is achieved by providing larger transition and connection section area than working section so that they remain elastic. The elastic modulus of working section will be change by, \( E_1 = \alpha E \) where \( \alpha \) is the factor of inelasticity. The equivalent stiffness will be expressed as Equation 8:

\[
K_e = \frac{1}{\frac{1}{E_1 A_1} + \frac{1}{E_2 A_2} + \frac{1}{E_3 A_3}}
\] (8)

2.2. Stability Analysis of Buckling Restrained Braces

Global Stability Analysis of Buckling Restrained Braces

Working basis of BRB is based on the fact that the restraining part will block the core section from buckling. If the restraining material is not strong enough, it will buckle together with the core and this will affect the global stability of the brace.

The ultimate strength (critical load \( P_{cr} \)) of BRB can be calculated from the Equation 12 deduced from Equations 9, 10 and 11 by assuming that the force is directly applied to core member as shown in Figure 3 and the friction between core member and restraining material is negligible [8] which means the force is transversally distributed and no longitudinal interaction.

![Figure 3. BRB free body diagram](image-url)
Equivalent equilibrium equation from free body diagram of Figures 3 and 4 is expressed by Equations 9 and 10 as:

$$E_1 I_1 \frac{d^4 y}{dx^4} + P \frac{d^2 y}{dx^2} = -q(x) \quad (9)$$

$$E_2 I_2 \frac{d^4 y}{dx^4} = q(x) \quad (10)$$

Where:
- $x$ and $y$ are axial and lateral displacement of BRB’s core component;
- $E_1, E_2$: Elastic modulus of core component and restraining material respectfully;
- $I_1, I_2$: moment of inertia of core component and restraining material respectfully;

By combining (9) and (10), $q(x)$ will be eliminated and Equation 11 will be deduced.

$$\frac{d^4 y}{dx^4} + \frac{p}{E_1 I_1 + E_2 I_2} \frac{d^2 y}{dx^2} = 0 \quad (11)$$

By introducing the boundary conditions at both ends and solving the Equation 11, the critical load of the overall stability can be obtained by taking the length of the restraining section and core as one.

$$P_{\text{cr, } g} = \frac{\pi^2 (E_1 I_1 + E_2 I_2)}{(\mu l)^2} \quad (12)$$

$\mu$ is the effective length coefficient.

The ultimate bearing capacity of the brace will be:

$$P_{\text{cr}} = \frac{\pi^2 (E_1 I_1 + E_2 I_2)}{(\mu l)^2}.$$ 

**Elastic-plastic Buckling Analysis of Brace Core Element**

The working principle for buckling restrained brace is that the load will be applied to the core material. When the load reaches the ultimate load, the restraining tube will restrain the core from excessive buckling.

Under strong earthquakes, if the overall bearing capacity of the brace is higher than the buckling load $F_t$ of the core section, the core will yield under compression before the overall instability occurs:

$$P_{\text{cr}} = \frac{\pi^2 (E_1 I_1 + E_0 I_0)}{(\mu l)^2} \geq F_t$$

$\mu$ is the length coefficient by considering the restraint conditions at the end of the brace. It’s value ranges between 0.7 and 1, 1 is considered in this paper.

**2.3. Analysis of Effect of BRB on the Stiffness of Frame Structure by Displacement-Based Method**

By considering single bracing frame shown in the Figure 5, the relation between the stiffness of the frame and that of the frame can be derived as follows:
The brace dimension for strengthening is deduced from displacement-based design of frame structures where, the displacement of the brace can be deduced geometrically from Figure 5 as the Equation 13 below:

$$ \Delta_{Brace} = \Delta \cos \theta $$

(13)

And by Equation 14 the axial force of the brace is related to the displacement by Hooke’s law as:

$$ F = \frac{EA}{L} \Delta_{Brace} $$

(14)

Therefore, the horizontal stiffness of the brace support is $$ K_{Brace} = \frac{EA}{h} \sin \theta (\cos \theta)^2 $$.

For brace with two members the horizontal stiffness will be twice that of single brace, Equation 15:

$$ K_{Brace} = 2 \frac{EA}{h} \sin \theta (\cos \theta)^2 $$

(15)

In this paper, the stiffness of the brace is computed by giving a target displacement that conform to the maximum elastic drift ratio and from that, all other parameters of the brace are deduced.

3. Analysis of Buckling Restrained Brace Parameters

In this paper Chinese codes are used as reference and the parameters of the BRB used in this paper have been analyzed and designed according to Chinese standard. The following are different parameters used:

3.1. Model

Bouc-Wen is a commonly used mechanical model for buckling restrained braces. The following is a brief introduction to the Bouc-Wen model [15, 16] used in this paper: The Bouc-Wen model can be described by Equation 16:

$$ F(x, z) = \gamma ku + (1 - \gamma)kz $$

(16)

Where, $$ \dot{z} = A\ddot{x} - q||X||Z||Z||n - 1 - \beta X||Z||n $$; $$ X, \dot{X}, \ddot{X} $$ are displacement, velocity and acceleration. $$ u(t) $$: External influence; $$ Z $$: Bouc-Wen hysteretic nonlinear restoring force. And $$ A, n, \alpha, \beta $$: are hysteretic constants, and the properties of hysteretic nonlinear restoring force $$ Z $$ which depend on material properties, response amplitudes and structural properties.

In the ETABS software, the default $$ A = 1, \alpha = \beta = 0.5 $$, and the larger the value of the exponent $$ n $$ is, the steeper the yield ratio is. If the $$ n $$ value is infinite, the Wen model is close to the bilinear model, but this may overestimate the energy consuming capacity of the damper. Therefore, $$ n $$ is chosen between 1 and 20 according to the hysteretic characteristics of components.

In this paper, Bouc-wen model is used as representative of Buckling restrained brace and its parameters are calculated from the material and geometrical characteristics of the braces used.

3.2. Bearing Capacity of Buckling Restrained Brace

In the seismic design, the axial bearing capacity design value of the buckling restraint support is determined according to Chinese code by Equation 17 from which yield and ultimate bearing capacity can be deduced to get representative Equations 18 and 19 respectively.
\[ N_b = 0.9A_1f_y \]  
\[ A_1 \text{ Cross section area of yielding core;} \]
\[ f_y \text{ Yield strength value of core element material;} \]
Yield bearing capacity is expressed as: \[ N_{by} = n_y A_1f_y. \]  
\[ n_y \text{: Yield strength coefficient of core element material.} \]

The ultimate bearing capacity which is used for designing joints is calculated by: \[ N_{bu} = \omega N_{by} \]  
\[ \omega \text{ Strain hardening adjustment factor for core element material.} \]

The yield strength coefficient and strain hardening adjustment factor for core element materials are dependent on the material type of the core element. Table (1) represents example of commonly used core element in buckling restrained braces and their respective characteristics.

### Table 1. Table yield strength, Yield strength coefficient and Strain hardening adjustment factor

<table>
<thead>
<tr>
<th>Material</th>
<th>( f_y ) (MPa)</th>
<th>( n_y )</th>
<th>( \omega )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q100LY</td>
<td>80</td>
<td>1.25</td>
<td>2.4</td>
</tr>
<tr>
<td>Q160LY</td>
<td>140</td>
<td>1.15</td>
<td>2.4</td>
</tr>
<tr>
<td>Q235</td>
<td>235</td>
<td>1.15</td>
<td>1.5</td>
</tr>
<tr>
<td>Q345</td>
<td>345</td>
<td>1.1</td>
<td>1.5</td>
</tr>
</tbody>
</table>

3.3. Equivalent Cross Section Area of Buckling Restrained Brace

In reality, buckling restrained brace is an assemblage of different parts longitudinally and transversally. Longitudinally, it is divided into connection zone, transition zone and yielding zone. The cross-section areas of these different zones are different and it is very complicated to consider them as different areas during simulation. Also, transversally, the BRB has core, the restraining material. In design, the equivalent cross section area of the BRB is computed by using the Table 2 which gives the relationship between the length of the brace, cross section area of the core brace and the equivalent cross section area.

### Table 2. Relation between cross section area \( A_1 \) of core element and equivalent area \( A_e \)

<table>
<thead>
<tr>
<th>Length of BRB</th>
<th>( A_1/A_e )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( L \leq 3 )</td>
<td>0.85</td>
</tr>
<tr>
<td>( L = 6 )</td>
<td>0.9</td>
</tr>
<tr>
<td>( L = 9 )</td>
<td>0.95</td>
</tr>
<tr>
<td>( L \geq 3 )</td>
<td>0.99</td>
</tr>
</tbody>
</table>

Relation between cross section area \( A_1 \) of core element and equivalent area \( A_e \) of Buckling Restrained Brace according to buckling restrained brace manual fourth edition of Tongji University.

3.4. Bending Stiffness Requirement of Encasing Element

In order to ensure that buckling restrained braces do not affect overall instability under earthquake, the flexural stiffness of encasing elements should meet the following requirements (Table 3) according to buckling restrained brace manual fourth edition of Tongji University. In this paper, energy dissipation BRB are used:

### Table 3. Relation between BRB type and encasing material stiffness

<table>
<thead>
<tr>
<th>BRB Type</th>
<th>Encasing material stiffness requirement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load Bearing BRB</td>
<td>( \frac{\pi^2EI}{L^2} \geq 1.2N_{by} )</td>
</tr>
<tr>
<td>Energy dissipation BRB</td>
<td>( \frac{\pi^2EI}{L^2} \geq 1.2N_{bu} )</td>
</tr>
</tbody>
</table>

\( E \): Elastic modulus of encasing material,  
\( I \): Weak axis moment of inertia of encasing material,  
\( L \): Length of BRB,  
\( N_{bu} \): Ultimate bearing capacity of BRB,  
\( N_{by} \): Yield bearing capacity of BRB.
4. Real Engineering Case Study

4.1. Description of Engineering Model

A three-story reinforced concrete frame building (Figure 6) with 16.5 meters height, built in 2008 is used as the model example for this paper. The building was designed according to Chinese codes. After construction, a heavy machine that has not been considered during analysis and design stage was installed to the rooftop and another floor is added to cover the machine, so this called for rechecking and strengthening of the original structure against additional loads.

The building is located in seismic fortification intensity of 7 and seismic acceleration of 0.1g and structural design service life of the building is 50 years. The building has columns and beams with different dimensions with largest columns of 700×700 mm and largest beam of 350×950 mm, the slab has dimension of 200 mm. All the dimensions from engineering drawings are respected in modeling of the structure all with concrete of C30.

![Figure 6. Engineering Model (ETABs)](image)

4.2. Structural Diagnosis of Engineering Model

By response spectrum analysis of the engineering model, the maximum displacement, maximum drifts and frame stiffness are computed using ETABS software [17]. Three modal shapes that are translation in X direction (Figure 7.a), translation Y direction (Figure 7.b) and rotation (Figure 7.c) show that the maximum displacement is on the last floor.

Weak areas are obtained by comparing results with Code for Seismic Design of Buildings” GB50011-2010 that stipulates that the maximum drift ratio must be less than 1/550, the maximum period ratio (ratio of the third mode period to the first mode period) must be less than 0.9 and the story stiffness of the floor must not be less than the maximum of 70% of the upper floor stiffness and 80% of the average of all above floors). The modal results (Table 4, Figure 7) shows that the building meets the vibration requirements according to the code.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Period(seconds)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.866</td>
</tr>
<tr>
<td>2</td>
<td>0.748</td>
</tr>
<tr>
<td>3</td>
<td>0.7</td>
</tr>
</tbody>
</table>

The period ratio of the structure is 0.8 and meets the vibration requirements. The story displacement results (Table 5), maximum story drift (Table 6) and the story stiffness (Table 7) are shown below.

<table>
<thead>
<tr>
<th>Story</th>
<th>Elevation (m)</th>
<th>Location</th>
<th>X-Dir (mm)</th>
<th>Y-Dir (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Story4</td>
<td>16.5</td>
<td>Top</td>
<td>19.185</td>
<td>15.041</td>
</tr>
<tr>
<td>Story3</td>
<td>12</td>
<td>Top</td>
<td>16.62</td>
<td>13.066</td>
</tr>
<tr>
<td>Story2</td>
<td>9</td>
<td>Top</td>
<td>12.671</td>
<td>10.851</td>
</tr>
<tr>
<td>Story1</td>
<td>5.8</td>
<td>Top</td>
<td>9.569</td>
<td>7.365</td>
</tr>
<tr>
<td>Base</td>
<td>0</td>
<td>Top</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>
According to elastic response spectrum analysis results, it is found that the frame structure will suffer from weak first story which lead to concentrated deformation to the first story with the horizontal stiffness that does not conform to the Chinese standard GB50011-2010. This is due to the fact that the first story does not have sufficient lateral bearing elements because of the intended use of the structure.
Also, the third story does not conform to the requirement of elastic story drift by the standard and this will make the building fail before entering to elastic plastic stage. This is due to the additional load and one story that caused the additional of the total mass of the building.

Therefore, it is necessary to reinforce the frame structure to improve the stiffness of the flexible floor and reduce the lateral displacement of the structure.

4.3. Reinforcement Scheme

Location of BRB

The braces must be arranged where they will solve the reinforcement objective without affecting the building function, stability and comfort of the users. The arrangement of BRB is done by the following three principles:

1. Where there is greater displacement and the resulting internal forces are greater, but without destroying the regularity of the original structure.

2. Where they will be insurance of the maximum possible symmetricity of the whole building and the stiffness center must coincide with mass center as much as possible to avoid torsion effect.

Because the earthquake loads are transmitted from the ground to the upper floors and its intensity is higher to the bottom than the top, reinforcing the bottom with energy dissipation devices will reduce the seismic energy transmitted to the upper floor of the building.

Also, when the reinforcement of the floor other than the first is needed (Third floor for this paper), braces should be arranged continuously from the bottom to the story where reinforcement is needed.

From the analysis results it is found that greater internal forces and displacement are located on the external joints in X and Y directions, therefore BRB are proposed to be located between external columns (Figure 9.b).

Because the stiffness of the first floor in all directions does not conform to the standards, braces are symmetrically proposed in all directions of the first floor (Figure 9.a).

For the third floor, braces are installed where there is maximum displacement at the same time avoiding vertical and torsional irregularity (Figure 9.b).

In order to respect the vertical regularity, the reinforcement of the third floor are provided at the second floor too (Figure 9.b) and in order to conserve the torsional regularity, all the braces are placed symmetrically with respect to X and Y directions.
In this paper, four different brace configurations were used, those are two chevrons (V and Inverted V), X configuration and Single brace.

**BRB Parameters**

In this paper, the buckling restrained brace has been simulated using ETABS 2017 and their length defined by frame dimension. Energy dissipation BRB was chosen for this paper and authors used Q235 as core material. The length of the brace was defined by the size of the other frame members where it was to be installed. By giving the first floor a maximum target displacement of 10.54 mm to conform to the standard of maximum drift ratio of 1/550, the displacement of the brace is computed as 7.4529 mm. In this paper, Q235 is used as core material for brace element and by using Tables 1 and 2, equivalence areas of the braces used are computed and presented in Table 8.

**Table 8. BRB parameters**

<table>
<thead>
<tr>
<th>Brace</th>
<th>Equivalent Area (cm²)</th>
<th>Material</th>
<th>Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>153.16</td>
<td>Q235</td>
<td>Energy dissipation</td>
</tr>
<tr>
<td>2</td>
<td>68.67</td>
<td>Q235</td>
<td>Energy dissipation</td>
</tr>
<tr>
<td>3</td>
<td>53.1</td>
<td>Q235</td>
<td>Energy dissipation</td>
</tr>
</tbody>
</table>

**4.4. Analysis Results of Strengthened Engineering Model**

The strengthened building was analyzed by response spectrum analysis under the same loading conditions as the original frame, and the following response characteristics are obtained.

**Modal Analysis**

**Table 9. Periods of first three modes of strengthened frame schemes (in seconds)**

<table>
<thead>
<tr>
<th>Model</th>
<th>SINGLE BRB</th>
<th>X BRB frame</th>
<th>INVERTED V BRB Frame</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.498</td>
<td>0.415</td>
<td>0.48</td>
</tr>
<tr>
<td>2</td>
<td>0.468</td>
<td>0.401</td>
<td>0.438</td>
</tr>
<tr>
<td>3</td>
<td>0.356</td>
<td>0.30</td>
<td>0.33</td>
</tr>
</tbody>
</table>
It can be seen from the Table 9 that there is a continuous vibration of the structure for both models including strengthened and original frames. However, the vibration period of single brace scheme is larger than other schemes.

**Rigidity Analysis**

It can be seen from the below graphs (Figure 10) that the overall stiffness of the structure strengthened by BRB has increased and the vibration period has decreased. The period ratios of the both X and V BRB scheme are less than 0.9 which meet the required condition of the torsional stiffness of the structure to be less than the lateral deformation stiffness.

The X direction stiffness (Figure 10.a) and Y direction stiffness (Figure 10.b) of the frame structure strengthened by X BRB configurations have greatly increased compared to other configurations.

![Figure 10. Stiffness of original binding and strengthened building](image-url)
Lateral Deformation Analysis

The results show that the four BRB configurations were able to strengthen the building conforming to the Chinese design code.

Lateral displacement of the reinforced concrete frame structure strengthened by X BRB (Figures 11.a and 7.b) decreases the most compared to other configuration types. And single BRB strengthened frame has larger drift ratio which means that even though all the BRB reinforcement configurations conforms with the standard Single BRB has is not a good choice compared to others.

According to China's seismic code for buildings, the inter-story drift ratio of reinforced concrete frame structures should not be greater than $1/550$ (about 0.00182) under frequent earthquakes. From the following Figures 11.c and 11.d, it can be seen that the drift ratio of the strengthened building meets the requirements. The drift ratio of the structure strengthened by X, V and single BRB meets the requirements of the corresponding design specifications with the drift ratio for X BRB configuration being the smallest.

The stiffness of the frame reinforced by single brace configuration is smaller than others, that means that the single brace still have an advantage of little increase of the stiffness of the frame structure after reinforcement, say the mass of the strengthened structure compared to other configurations. For the tallest building where the top most stories are to be strengthened, this single BRB configuration may be beneficial in order to reduce the seismic effect of the strengthened structure.

(a) Displacement X-Dir

(b) Displacement Y-Dir
4.5. Comparison between Ordinary Braces and BRB Strengthened R-C Frame Structure

The researchers have demonstrated that lateral stiffness of frame structures can be improved by both ordinary braces and buckling restrained braces by setting them up to flexible stories. According to literatures, the following are differences between ordinary braces and BRB are highlighted as follows:

The bearing capacity of ordinary braces are determined by

\[ N_b = \frac{\varphi A f}{1 + 0.35 \lambda_n} \]

And that of BRB is determined by \( N_b = 0.9A f_y \).

With, \( \varphi \):stability coefficient of compression members; \( A \): The cross-section area of the brace; \( \lambda_n \): Adjusted slenderness ratio of the brace, \( \lambda_n = \left( \frac{A}{\pi} \right) \left( \frac{f_y}{E} \right) \); \( \lambda \): Slenderness ratio of the brace; \( f_y \): Yield strength of the steel; \( E \): elastic modulus of the steel.

Stability coefficient and slenderness ratio are essential in design of ordinary brace. According to GB50017-2003, for class a and class b Q235 steel, stability coefficient is less or equal to 1. Therefore, by comparing the two formulas of elastic bearing capacity, when the same steel material and same equivalent areas are considered for both ordinary and buckling restrained braces, the bearing capacity of buckling restrained brace will be larger than that of ordinary brace. By equalizing the two formulas for bearing capacity and assuming that the same material is used for both buckling restrained braces and BRB and taking the maximum value of stability coefficient, the required area of ordinary brace to the same bearing capacity as BRB is found to be 1.215 times that of BRB.
In order to effectively compare seismic effect of the two types of braces, ordinary brace with the same configuration (X type), material and dimensions as BRB are installed at the same location in reinforced concrete frame structure as BRB.

(a) Story displacement X-Dir

(b) Story Displacement Y-Dir

(c) Story Drift X-Dir
From the results above (Figure 12), it can be seen that under action of small earthquakes the reinforced concrete frame with ordinary braces also satisfy the maximum drift ratio (Figure 12.c, d) and displacement (Figure 12.c, d) requirements for Chinese building seismic code.

The interlayer displacement of the frame structure strengthened by ordinary braces and BRB both conforms to the requirements of the specification, but the drift for structure strengthened by ordinary braces is larger than that of BRB brace (Figure 12.c, d). This is because the same cross-section areas are considered in this paper.

Because during small earthquakes, all braces are supposed to remain in elastic state, when the ordinary braces are designed according to the Chinese requirements, they will have bigger area than BRB and consequently high stiffness and this will result in smaller drift ratio and building horizontal displacement that of BRB. Therefore, it can be recommended when the cost of the project is not a one of the selection criteria because the larger cross section area of ordinary brace will be costly.

When only rigidity and bearing capacity is to be improved, both BRB and ordinary brace can be used but the first can be economical and the last have better rigidity impact.

4.6. Non-linear Time History Analysis of Strengthened Structure

During elastic response spectrum analysis, the building response is assumed to respond in exclusively elastic manner, but because of the geometrical non linearity of the building, material non linearity of some structural members and possible seismic non linearity behaviors of some structural members, it is beneficial to perform nonlinear analysis.

In this paper nonlinear time history analysis of the strengthened building is analyzed under strong earthquakes.

Selection of Seismic Waves

According to the Code for Seismic Design of Buildings in China, the actual five strong earthquake records and two synthetic earthquakes are selected according to the types of building sites and design earthquakes grouping.

The spectral characteristics of the selected seismic waves were close as possible to the characteristic period of the building site, and the duration of the seismic waves selected conforming to the code.

The strengthened building was checked against strong earthquakes and the two types of braces are compared according to joint displacement, acceleration and base shear.

Result Analysis

The results show that under rare earthquakes, the base shear (Figure 13.c), peak acceleration (Figure 13.b) and peak displacement time history (Figure 13.a) of BRB structure are smaller than those of ordinary braces braces. The buckling restrained braces provide additional damping ratio for the structure, which reduces the displacement response of the structure under earthquakes and reduces the damage of the main structure caused by earthquakes.
Figure 13. Time history results of Building strengthened by BRB and by ordinary braces
5. Conclusion

The results of elastic response spectrum analysis show that under action of small earthquakes, both buckling restrained braces and ordinary braces can be used for strengthening of reinforced concrete frame structures according to Chinese requirement of seismic design, GB 50011-2010. This is due to the fact that ordinary braces will not buckle during small earthquakes. When both brace types are compared, ordinary braces will need to have the cross-section area that is bigger than that of BRB in order to have the same stiffness performance.

By performing nonlinear time history analysis, more ordinary braces fail due to excessive buckling but buckling restrained braces remain stable, this is shown by good performance of the frame structure restrained by buckling restrained braces compared to that of ordinary braces. One can ordinary brace is not a safe alternative to be used in strengthening of concrete frame structures where strong earthquakes are expected.

Different configurations of BRB are studied. The results show that inverted V buckling restrained braces are better than V BRB, this is explained by the fact that for inverted V brace when one member is under tension another is under compression, and the force is directly transmitted to the column of the next lower floor. But for V brace, the load will be transmitted to the beam then to the column and this will affect the bearing capacity.

6. Conflicts of Interest

The authors declare no conflict of interest.

7. References


The Fire Exposure Effect on Hybrid Reinforced Reactive Powder Concrete Columns

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Abstract

This paper offers an experimental investigation of the fiber reinforced reactive powder concrete columns' behavior after exposure to fire and improvements made to improve column resistance against fire. This study is mainly aimed to study the experimental behavior of hybrid reinforced columns produced by reactive concrete powder (RPC) and exposure to the flame of fire at one side and subjected to eccentric load. The experimental methodology consists of sixteen RC columns that organized into four groups based on the variables used in this research: (SF) steel fibers, (PP) polypropylene fibers, (HB) hybrid fibers, (PPC-SF) hybrid cross-section (steel fiber reactive powder concrete core with polypropylene fiber reactive powder concrete cover). All columns were tested under 60 mm eccentric load and the burn columns were exposed to fire for different duration (1, 1.5 and 2) hours. The results indicated that (SF-RPC, PP-RPC, HB-RPC, PPC-SFRPC) columns exposed to a fire flame for the period 2 hours, lost from their load capacity by about (54.39, 40.03, 34.69 and 30.68) % respectively. The main conclusion of this paper is that the best fire resistance of the column obtained when using a hybrid cross-section (steel fiber reactive powder concrete core with polypropylene fiber reactive powder concrete cover).

Keywords: Reactive Powder Concrete (RPC); Hybrid Cross Section Column; Hybrid Fibers (HB); Exposed to Fire and Eccentric Load.

1. Introduction

The reinforced concrete column is a structural member utilized mainly for standing compressive loads, consisting of concrete with an embedded steel frame for reinforcement purposes. There are rarely axially loaded columns in practice since there is always some bending. The moments that happened in the continuous construction along with unavoidable building imperfections will cause eccentricities and then caused a bending in the member. The strength of the column is controlled by the strength of the used material (in particular, the compression strength of the concrete) and the cross-section geometry [1]. The demand for stronger, products with lower space-consuming has increased as construction and material costs increase. Newly, in Bouygues, France, developed a very high strength and high ductility cement-based composite, known as reactive powder concrete (RPC) [2]. RPC is a cemented material characterized by high-performance characteristics for example low shrinkage creep and permeability, ultra-high strength and increased protection against corrosion [3]. However, the need for high-strength structures always comes with an issue in fire resistance for the structure. It was disclosed collectively that the greater strength of the blend will cause a reduction in the composition's fire resistance. In high temperature, the high-performance concrete compositions which are usually denser tend to be more likely to fail because of their high brittleness. High performance concrete shows greater deterioration than ordinary strength concrete, for example concrete spalling and cracking [4]. Nowadays, many fire accidents have occurred around the world, with the use of fresh cement developments (lately RPC) to build load-carrying...
members for high-rise structures composed of beams and columns, and the fire safety design of these structures has become crucial. This is because the fire resistance of these members is the recent line defense, if other means is failed in extinguish the fire [5]. Also, secure constructions must be designed with a minimum danger for both individuals and property as potential [6, 7].

Nevertheless, the previous study concentrated only on the efficiency of the concrete columns during the fire, whereas the performance of these columns after cooling was very crucial since most concrete buildings subjected to fire circumstances did not collapse and could be recycled using appropriate methods for repairing [8]. In spite of that, it is not easy to decide whether it is more economical to repair the fire-exposed buildings or to demolish and repair them. This choice requires a full understanding of the conduct of these constructions after exposure to fire to determine whether the residual load-bearing capability of the load-bearing members is adequate. The previous researches indicated that the main cause of the crash was steel reinforcement failure for most of the concrete buildings that were damaged by fire [9, 10]. The reason is that the position of the reinforcement is generally near to the surface of the concrete member. Therefore, the steel reinforcement initially deteriorates due to its higher transfer rate of heat compared to the concrete [10]. The tests were carried out as the most critical situation on four sides fired concrete columns. Concrete columns may be exposed to fire from various sides in actual fire events based on the construction’s architecture and structural design [11, 12]. For instance, a wall could operate as a column obstacle exposing only one, two or three sides of the column to fire. On the other side, a column can be situated in the center of a space thus exposing all four sides of the column to fire.

In this research improvements made to improve column resistance against fire. One of the improvements is to make a model in which the main component of the column Steel fiber strengthened RPC (SF-RPC) in the (core) and protected using polypropylene fiber strengthened RPC (PP-RPC) in the (cover). In this case, the core column is not significantly affected, the designer that can used after burning by the rehabilitation of the column through maintenance on the cover only. Also hybrid fibers are used in optimal proportions to differ from previous researches columns to obtain a column with elevated burn resistance and it has compared to columns casted, containing only steel fibers or polypropylene fibers. Figure 1 show the research methodology.

Figure 1. Flow chart of the project of research
2. Experimental Program

2.1. Material

2.1.1. Cement

The cement that utilized in this study is Ordinary Portland cement (Type I), cement produced in Iraqi north and known of commercially as Karasta.

2.1.2. Fine Aggregate

Very fine sand with maximum particle size (600 μm) was utilized as a fine aggregate for reactive powder. In compatible with Iraqi Specification requirements IQS No.45/1984. Figure 2 shows the natural sand grading curve.

2.1.3. Micro Silica Fume

RPC mixtures contain densified micro silica which is a pozzolanic material with particle size ranging from 0.1 to 1 μm. In terms of particle diameter, the average particle size is about 100 times smaller compared to Portland cement. The silica fume is manufactured in accordance with the requirements (ASTM C 1240/2005).

2.1.4. Fibers

In this research, uses of two kinds of fibers, Micro Steel Fiber (SF) and Micro Polypropylene Fibers (PP) having (13±1) and (12) mm length, (0.2±0.05) and (0.032) mm diameter, (2400) and (600-700) MPa Tensile Strength, (7825) and (910) Kg/m³ density respectively.

Figure 2. Grading curves for fine sand

Figure 3. Sample of Micro polypropylene and steel fibers
2.1.5. High-range Water Reducing Admixture (HRWRA)

In this research, superplasticizer type (Hyperplast PC200) conform with (ASTM C494, 2017, Type, G, F) was used which is chloride-free polycarboxylic polymers with long chains.

2.1.6. Steel Reinforcement Bars

There are three types of steel reinforcing bars used in this research: First: deformed steel bar Ø8 mm nominal diameter within all columns as longitudinal reinforcing (Ukrainian production). Second: deformed steel bar Ø10mm nominal diameter within all corbels reinforcement. (Ukrainian production). Third: deformed steel bar Ø6mm nominal diameter as transverse ties in the corbels and columns. (Turkish production).

2.2. Concrete Mix Design

Based on the previous researches, many trials RPC mixes were made to gain the higher compressive strength and flow (110%±5) according to (ASTM C109, 2016 and ASTM C1437, 2015) respectively. In the present research, three kinds of RPC mixes were used, as shown in the table (1). The variable used in these mixes was, fiber type (steel fibers, polypropylene fibers and hybrid fibers).

<table>
<thead>
<tr>
<th>Mix No.</th>
<th>Mix ID</th>
<th>Mix Proportions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Cement Kg/m³</td>
</tr>
<tr>
<td>1</td>
<td>SF-RPC</td>
<td>950</td>
</tr>
<tr>
<td>2</td>
<td>PP-RPC</td>
<td>950</td>
</tr>
<tr>
<td>3</td>
<td>HB-RPC</td>
<td>950</td>
</tr>
</tbody>
</table>

2.3. Mixing Procedures

The specimens’ mixes were carried out in a pan mixer of (0.09m³) for RPC. In this research, RPC was produce in a simple method without accelerated cure systems to simulate the practical site conditions. At the beginning of the mixing process, silica fumes and fine sand were mixed for 3 minutes, then for 2 minutes the cement and dry components were mixed until the uniform color was obtained. The superplasticizer was added to the water then mixed liquid was added to the dry mix for another 4 minutes during the mixer rotation. Fibers were lastly added by hand within 5 minutes during blending.

2.4. Specimens Details

All the columns have the same size and nominal dimensions. The model dimensions were a square section of 120×120mm and 1000 mm total length and 500 mm length localize between the corbel (middle part). The constant eccentricity value e = (60) was assumed. The corbels were reinforced by additional steel bars to avoid early failure in this part of the specimens during the tests and to concentrate the failure in mid part, as shown in Figure 4. Depending on the following variables, the columns were organized into four groups: steel fibers, polypropylene fibers, hybrid fibers and hybrid cross-section steel fiber reactive powder concrete core with polypropylene fiber reactive powder concrete cover. Each group consists of four reinforced reactive powder concrete columns. The test program and specimen details are summarized in Table 2. Symbols defined each column are (SF) refers to the steel fiber, (PP) refers to polypropylene Fiber, (HB) refers to hybrid fibers mixture (Steel fiber polypropylene Fiber), (RPC) refer to reactive powder concrete, (PPC) refer to use polypropylene fiber in cover and (SFPC) refer to use the steel fiber in the core. The symbol (1, 1.5 and 2) refers to the duration of fire exposure (hour).
2.5. Casting Procedures

Steel molds were used. The fresh concrete was placed in three layers of column specimens (SF_RPC, PP_RPC, and HB_RPC). After putting each layer, the vibrator was used to compact the layer and ensure that the trapped air was removed and left for 24 hours until it was hardened. The samples were got out of the molds and cured with saturated wet coverings using burlap but column specimens (PPC-SFRPC) were cast in a different method: the first stage, core and corbel of the concrete mixed column cast (SF_RPC) and polystyrene foam sheet put surrounding the reinforced of the column to prevent concrete from escaping to the cover. The vibrating table was used during casting. After 24 hours, the polystyrene foam sheet was removed, then the surface of the core was cleaned and intentionally roughened to facilitate the process of bonding the new casting concrete in the after stage. The prepared sample was returned to the steel mold and casting the empty cover with PP_RPC in addition to the use of the vibrating table as mentioned before.
During the cover casting, the corbel was covering by saturated wet burlap. After 24 hours, the specimen got out of the mold and cured with saturated wet burlap, as shown in Figure 4 (a, b & c).

![Images of hybrid section column reinforcement, hybrid section mold and steel reinforcement, and hybrid section after casting](image)

**Figure 4. Method casting hybrid section columns**

### 2.6. Burning Process

At the college of engineering in the University of Babylon, the laboratory of concrete research, material properties and fire exposure studies were conducted there. Figure 5 (a) illustrate Furnace and equipment used in the burning process. The main structure of the furnace was produced from refractory brick and refractory mortar in addition to the presence of a small opening to provide adequate air for the burners. The main objective of the furnace compartment is to increase the temperature constantly for a required period for each type of columns as shown in Figure 5. The burner’s network made from three methane burners, which organized in one lines at one side of exposure and their distribution were all over the column length without corbels, methane burners were connected together in one pipeline. The equipment that controlled the fire temperature and the heating process were shown in figure 5. At the age of 28 days, the curing process was finished, after that, the samples will be left for 26 days without curing and in the day 55th the will subject to burn, then the test carried out in (56 days age). The part of the concrete specimen that localized in between corbels (middle portion) were fair-faced toward methane burners. After the duration of burning was achieved (1, 1.5 or 2 hours of burning) the gas valve closed. After that, it left the specimens to cool down to the ambient temperature.

### 2.7. Testing Process

All RPC column specimens were tested at the age of 56 days under a calibrated electro-hydraulic testing machine with (capacity of 650 KN). An eccentricity distance of 60 mm was used in the present work. The load was applied at a loading rate of 1 kN/s, using load control. For each increment in the load, the lateral deformation at mid-height and axial displacement were measured by two dial gauges with an accuracy of 0.01 mm per deviation and a capability of 50 mm. One dial gauge was placed on the testing machine piston and the other at the mid-height place along the vertical centerline of columns. Testing continues until failure recorded by a drop in load record.
Figure 5. All details of the furnace and equipment and a time-temperature curve

3. Results and Discussion

The experimental results for the columns of each group are compared to each other to determine the effect of duration of fire exposure (1, 1.5 and 2 hours) then compare each group with another group to investigate the effect of fiber type. The test results of the specimen were the ultimate load carrying capacity, the first crack load, load- lateral deflection relationship, axial deformation, the cracks generated with a load of the columns, and failure mode of column samples.

<table>
<thead>
<tr>
<th>GROUP NO.</th>
<th>Columns Symbol</th>
<th>Fire Duration (hr.)</th>
<th>Load Carrying Capacity Pu (kN)</th>
<th>Percentage Decreasing Load Carrying Capacity (%)</th>
<th>Ultimate axial deformation (mm)</th>
<th>Ultimate mid-height lateral deflection (mm)</th>
<th>First crack load Pcr (KN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ONE</td>
<td>SF-RPC</td>
<td>0</td>
<td>320.36</td>
<td>Reference</td>
<td>7.78</td>
<td>9.9</td>
<td>83</td>
</tr>
<tr>
<td></td>
<td>SF-RPC-1</td>
<td>1</td>
<td>290.03</td>
<td>9.46</td>
<td>7.79</td>
<td>9</td>
<td>66</td>
</tr>
<tr>
<td></td>
<td>SF-RPC-1.5</td>
<td>1.5</td>
<td>166.34</td>
<td>48.07</td>
<td>6.41</td>
<td>8.46</td>
<td>40</td>
</tr>
<tr>
<td></td>
<td>SF-RPC-2</td>
<td>2</td>
<td>146.11</td>
<td>54.39</td>
<td>6.3</td>
<td>8.85</td>
<td>36</td>
</tr>
</tbody>
</table>
Pureack has been improved. The first crack load of RPC columns with steel for fire periods. These fibers melt and form channels to fiber group three and four.

3.1. Load Carrying Capacity

The load carrying capacity reflected the ultimate load applied to tested column specimens after which a decrease in device reading occurred with a fast column deformation known as a failure. The behavior of columns tested at ambient temperature as a reference specimen for comparison to study the loading capacity remaining of columns after burning. As stated before, the columns are subdivided into groups. The results showed that the non-exposed control column of group one has more load capacity than the control column of group two while group three and four for the control column have load carrying capacity close to control column of group one. As for the columns exposed to fire for duration of fire exposure 1 hour for each group, the decrease in the load carrying capacity of (SF-RPC, PB-RPC, HB-RPC, PPC-SFRPC) columns (9.46%, 4.53%, 18.39% and 13.81%) respectively from reference in each group but the decrease in the column load carrying capacity exposed to fire for duration of fire exposure (1.5 and 2) hours for each group (SF-RPC, PB-RPC, HB-RPC, and PPC-SFRPC) columns are (48.07%, 21.14%, 22.55%, and 21.28%) and (54.39%, 40.03%, 34.69% and 30.68%) respectively from reference column in each group as shown in figure 6. Adding polypropylene fibers considerably decreases RPC spalling during fire periods. These fibers melt and form channels to escape water vapor to relieve the inner vapor pressure of the RPC while the existence of these channels decreases the RPC strength. This is confirmed in many types of researches. However, hybrid fiber reinforced reactive powder have a more stable behavior under fire exposure and the greater residual load carrying capacity.

This is because the hybrid fiber-reinforced RPC had preferable resistance to elevated temperatures compared to (steel or polypropylene) fiber-reinforced RPC as well as a successful way to improve the resistance of RPC to explosive spilling is to add hybrid fiber (steel fiber and polypropylene fiber), which should be a main objective for improved fire resistance.

3.2. Load-Displacements Relationship of Reinforced Concrete Columns

From Figure 8 (a, b, c & d), it could be observed that the rise in duration of column fire exposure has an important impact on the mid-height lateral deflection of column specimens for all groups. Also, this comparison means that load-deflection curves for group (two and three) are more sensitive to high temperatures compared with a group (one and four). This can be ascribed to the reality that heating causes a decrease in the stiffness of the column owing to the decrease of the concrete elasticity modulus and the decrease in the effective section due to cracking.

3.3. First Crack Load

The cracks formed in the concrete if the tensile stress extents to its strength limit at the interface of cover-core. After the cover-core interface cracks advanced, the concrete cover was free to spall or buckle away. In all tested columns, the first crack was experimentally measured, in this research, the first experimental crack load for columns was visually observed by a magnifying lens while the load was gradually increased. The first crack was seen in all samples at mid-height of the column on the tension face (maximum moment region). Table 3 shows this. With growing compressive strength of concrete and fiber type, this first crack has been improved. The first crack load of RPC columns with steel and hybrid fibers, compared to RPC columns with polypropylene fibers was better by (3-2.5) times. Hence, the steel fibers are better to be used to enhance the tension, split and rupture strength of concrete. In general, the first crack load of RPC columns reductions with increasing the fire exposure’s duration.

<table>
<thead>
<tr>
<th>GROUP</th>
<th>PB-RPC</th>
<th>PP-RPC-1</th>
<th>PP-RPC-1.5</th>
<th>PP-RPC-2</th>
<th>HB-RPC</th>
<th>HB-RPC-1</th>
<th>HB-RPC-1.5</th>
<th>HB-RPC-2</th>
<th>PPC-SFRPC</th>
<th>PPC-SFRPC-1</th>
<th>PPC-SFRPC-1.5</th>
<th>PPC-SFRPC-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>TWO</td>
<td>0</td>
<td>238.29</td>
<td>227.48</td>
<td>187.9</td>
<td>142.9</td>
<td>315.10</td>
<td>257.15</td>
<td>224.03</td>
<td>205.77</td>
<td>294.74</td>
<td>254.03</td>
<td>204.29</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Reference</td>
<td>4.53</td>
<td>21.14</td>
<td>40.03</td>
<td>Reference</td>
<td>18.39</td>
<td>22.55</td>
<td>34.69</td>
<td>Reference</td>
<td>13.81</td>
<td>21.28</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>5.32</td>
<td>4.84</td>
<td>5.8</td>
<td></td>
<td>7.03</td>
<td>8.2</td>
<td>8.22</td>
<td></td>
<td>5.77</td>
<td>5.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>9.74</td>
<td>10.4</td>
<td>17.45</td>
<td></td>
<td>11.66</td>
<td>13.85</td>
<td>15.30</td>
<td></td>
<td>14.75</td>
<td>13.44</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>28</td>
<td>26</td>
<td></td>
<td></td>
<td>52</td>
<td>47</td>
<td></td>
<td>87.37</td>
<td>60</td>
</tr>
</tbody>
</table>

3.70
4. Failure’s Mode

The failure mechanism of all columns was caused by yielding the longitudinal steel reinforcement, which occurs after buckling and cracking that happened at the tensile face of the specimen while the crushing and deboning at a compression face of unburned specimens. Generally, this was depend on concrete compressive strength and fiber type. The columns that contain steel fiber and hybrid fiber have high ductility in compare with columns that contain only polypropylene fiber. This affect the failure mode. Also, these fibers effected preventing the exploding and spalling concrete cover even after failure.

In burned columns, the presence and spreading of cracks are faster than unburned columns. In (SF-RPC-1, SF-RPC-1.5, SF-RPC-2) columns, a crack has been started in the side of the corner of the columns then spread towards the compression zone, in (SF-RPC-1) the concrete cover’s debonding and crushing was happened at this face, while there are explosive concrete spalling before testing columns in the (SF-RPC-1.5, SF-RPC-2) columns due to the high temperature in the compression face, so spread cracks and columns failure is fast [13]. The form of failure of (PP-RPC-1, PP-RPC-1.5, PP-RPC-2) columns almost similar to the failure of (SF-RPC-1, SF-RPC-1.5, SFRPC-2) but cracks appear faster and concrete spalling of the concrete cover occur after test at a compression zone in the middle third of the column. The failure mode of (HB-RPC-1, HB-RPC-1.5, HB-RPC-2) columns are similar to previous unexposed columns where no explosive concrete spalling happened before testing the columns due to high temperature and after testing columns due to high pressure at compression zone. The failure mode in PPC-RPC-1, PPC-RPC-1.5, PPC-RPC-2 is similar to (PP-RPC) that exposed to fire but the cracks spread slowly at tension face. In addition to thermal cracks due to high temperature before testing and concrete spalling after testing at compression zone in the middle third of the column, there are splitting along an area of junction between core and cover. As shown in Figure 9, the start of failure can be observed for all RPC columns by listening to the popping sound and pulling of steel fibers then gradually dropping down the load applied.

5. Ductility

In this paper, the method of (energy absorption capacity) is adopted to the study the ductility. The absorbed energy capacity of the concrete column defined as the enclosed area under the curve of load vs. displacement until the maximum load was reached [14]. The result of calculation of (ductility) is given in Figure 7. From these results and figure turns out, using steel fibers in reactive powder provide a significant enhancement for the ductility of an unexposed column (reference) in group one from other references for other groups. From these results, the residual ductility of columns for group one after burning are (93.45, 44.44, and 41.96) % to columns exposed for the duration of fire exposure (1, 1.5 and 2 hour) respectively. For columns of a group two, group three and group four (89.11, 67.96 and 74.52) %, (86.3, 84.27and 83.08) % and (84.71, 68.14 and 66.99) % for columns exposed for the duration of fire exposure (1, 1.5 and 2 hour) respectively.

Figure 6. The percentage of load carrying capacity of columns

Figure 7. The ductility of specimens
(a) Load–deflection of Columns for group one.

(b) Load–deflection of Columns for group two.

(c) Load–deflection of Columns for group three.

(d) Load–deflection of Columns for group four.

Figure 8. Load–deflection of Columns for each group
Figure 9. Failure modes of (HB-RPC), (PPC-SFRPC), (SF-RPC) and (PP-RPC) columns after the burning and testing

6. Conclusions

- For the reference RPC columns of each group, the load carrying capacity was higher when using steel fiber, hybrid fiber and hybrid cross section compared to using polypropylene fiber.

- The effect of the increased duration of fire exposure on the load carrying capacity was less for (HB-RPC, PPC-SFRPC) column specimens than (SF-RPC, PP-RPC) column specimens. For 2 hours fire duration the best residual load carrying capacity is obtained by using (HB-RPC and PPC-SFRPC).

- From the test results, it has been concluded that the cover of concrete contributed to improving the fire resistance of (PPC-SFRPC) columns after burning for different burning durations.

- For the reference columns, it is observed that SF-RPC columns have much greater ductility compared to other references in each group. Also, it is discovered that using polypropylene fiber in RPC such (PP-RPC) columns gives lower ductility compared to other groups.

- The experimental findings show that the absorption energy capacity of RPC columns clearly decreases with increment the duration of fire exposure.

- The first crack load of RPC columns with steel and hybrid fibres, compared to RPC columns with polypropylene fibers was improved to (3-2.5) times. The first crack load of RPC columns decrease with increasing the duration of fire exposure.
Hybrid, polypropylene and Steel fibers influence the prevention of explosion and spalling concrete cover even after column failure. However, columns (HB-RPC) have no explosive spalling occurred before testing the columns due to elevated temperature and after testing columns.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


Fatigue Resistance Models of Structural for Risk Based Inspection

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Abstract

The current stage of civil engineering is characterized by special attention to the safety of structures with a long service life. Such objects were designed several decades ago and their safe operation was ensured by significant safety margins. Now this approach to safety has been replaced by the concept of acceptable risk. It forms the basis of a risk based inspection (RBI) maintenance strategy. The transition from preventive maintenance strategies to a technical condition maintenance is substantiated. Complex indicators of technical condition, suitable for RBI-maintenance, are considered. The methodology of the resource safety index (RSI) is proposed. The latter is used as an indicator of risk. Special models of fatigue resistance is required for its control. The purpose of this paper is to build fatigue models for critical structural elements that are serviced according to the RBI concept. Instead of the traditional S-N curve, the lifetime general equation (first model) be used, where by the arguments are the main influence factors. Along with this, a modified ε - N equation is proposed for deformation criteria. The novelty of this equation is that it uses the rate of S-N curve (slope) obtained in the first model with high cycle fatigue. The second model, combining the results of fatigue tests, is the equation for the dispersion of durability. The third model is the accumulated damage function under overloads. The efficiency of the RSI method is demonstrated by the example of the reliability assessment of the high strength bolts. Thanks to RSI method forecasting, during RBI-maintenance, parts can be used 3-5 times longer than with traditional methods.

Keywords: Risk; Overload; Damage; Lifetime; Safety Index.

1. Introduction

At the contemporary stage of engineering development, fatigue resistance models have begun to be applied to objects whose safety was earlier considered in an absolutely static aspect. These objects began to include bridges, buildings, pipelines, supporting structures of industrial equipment. For example, the crash of a viaduct in Genoa (2018), scientists explain on the underestimated phenomenon of very-high cycle corrosion fatigue in existing civil infrastructures. The brittle destruction of the bridge’s cable triggered the collapse of the whole structure. The aggressive environment, as well as the structural size effect, both may change Wöhler’s curve (a model of resistance fatigue), translating it downwards and eliminating the horizontal asymptote at the basis of the concept of fatigue limit [1].

Structures whose failures are related with significant consequences can be designed with a large margin of safety. As a result, they have a long, but unfortunately an indefinite (uncertainty), period of operation. The final decision on the issue of service life, as a rule, falls to the stage of exploitation. By the beginning of the XXI century in the circles of specialists engaged in the service of industrial equipment, the look at maintenance was shaped as an integral part of enterprise [2]. The issues of service and repair of mechanical equipment always were important for the industrial sector.

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For example, in the total value of metallurgical products, more than half of the costs are related to the maintenance of equipment.

The share of repair personnel can reach up to 30% of the metallurgical enterprise staff. If the average share of maintenance costs for all costs in general industrial production is 5%, then this share for chemical production is 6.8%, and for steelmaking it is already 12.8%. In metallurgy, maintenance costs are 8.6% of investments in production (against 3.8% in chemistry) [2, 3].

In order to optimize expenses in this area, the management of enterprises gradually began to move away from the traditional preventive repair strategy (PM) with its rigid schedule. At some enterprises, the cramping in the funds for maintenance and repair led to a corrective maintenance strategy (CM), when repairs are made after failures. In both strategies, spending is far from optimal. The use of PM and CM-strategies can be justified only at certain stages of exploitation. Minimizing the costs of maintaining the equipment and maximizing its availability is facilitated by the technical condition maintenance (TCM) strategy. This strategy involves the use of a flexible schedule of repairs and is characterized by the active application of methods of technical diagnostics. Sometimes such strategies are referred to as proactive (predictive) maintenance [2, 3]; imperfect maintenance [4]. As a rule, TCM-servicing is performed by means of parameter monitoring. A type of strategy with the control of complex reliability indicators (Reliability centered maintenance - RCM) is also widely used [2]. There is already a task of assessing the consequences of failures, which leads to the creation of a strategy with risk and safety control (Risk based inspection concept - RBI). Considering the above, manufacturers of metallurgical equipment (for example, SMS group, Danieli, Eirich) began to equip it with built-in systems of monitoring technical condition. Typically, the automated system includes a subsystem for monitoring the torque of the drive and the subsystem of vibrodiagnostics of rotor assemblies. With their help, maintenance personnel receive recommendations on the dates of replacement of nodes. This trend is relevant, as a result of the new-fangled outsourcing from the equipment is deleted experts-mechanics who are constantly observing it. But the amount of information characterizing the technical condition increases and becomes more complicated.

The purpose of this paper is to build fatigue models for critical structural elements that are serviced according to the RBI concept. Such models should be suitable for assessing the current values of complex indicators of technical condition according to the indications of monitoring systems.

The aim of the paper includes:

- Development of a complex indicator of the technical condition for RBI maintenance;
- Directly the development of fatigue resistance models;
- Demonstration of the practical use of the developed algorithms.

2. Complex Indicators of Technical Condition

The probability of survival operation $P$ (PS) is used as a complex diagnostic indicator for mass production systems. In the classical formulation PS characterizes the relative number of failures. On the one hand, the algorithms for predicting this indicator should not be too sensitive to the growth of the number of elements of a technical system. The trend of growth in the number of calculated and diagnosed elements can lead to an unjustifiably low projected level of PS for the whole system. As a result, the cost of the object increases. On the other hand, the PS should react to the operating time if it acts as a diagnostic parameter. Both requirements contradict each other. This circumstance forces us to seek for better indicators.

Among them the risk $\rho$ is present, in which the probability of failure is ranked by the level of its criticality $n$. In this way, failures of the technical system are reduced to the same scale. But it is not possible to completely avoid the shortcomings inherent in PS. To avoid these contradictions, a Resource Safety Index is used, representing the logarithm of the guaranteed margin of lifetime $n_{SR}$ for safety $R$ [5]:

$$
\beta_R = \log n_{SR} = \log \frac{T_0}{t_R} = \log T_0 - \log t_R
$$

(1)

Where $T_0$ is the minimum longevity value obtained from the lifetime distribution function (LDF) for the failure probability $Q = 1-R$; $t_R$ = maximum value of the operating time, determined from the distribution function of the operating time for the probability $R \geq 0.5$.

As a result of the use of the Resource Safety Index, the kinetic model of the degradation process such as depletion of the resource (decreasing function) is divided into two sections. At the initial stage of operation, the technical condition is monitored according to the safety index diagram, which is in double logarithmic coordinates by a straight line (Figure 1).
Figure 1. Scheme of the degradation process model based on the safety index for a (a) simple and (b) complex technical system

It leaves the point $\lg T_0 = \beta_{R0}$ and falls into a point with the same lifetime for $\beta_R = 0$. Controlling the operating time $t$ of the most dangerous place exposed to the most intense damaging process (the weak link principle; simple system), when approaching $\beta_R \rightarrow 0$, and $\beta_r \rightarrow T_0$, it is advisable to assign an inspection of the object and control of the technical state (check, Figure 1). For $\beta_R > 0$, the object is in an operable state (G, Figure 1). At $\beta_R < 0$, the technical state is controlled not by the safety index but by the reliability function $P(t)$, since in this region it is sensitive to the running time. At $P > 0.5$, the object falls into the zone of preventive replacements (reparations) (PM, Figure 1). During further operation, when the probability of survival operation drops to $P < 0.5$, and the risk rises to $\rho \rightarrow 1$, the object is in the zone of emergency failures (CM, Figure 1).

The transition to the method of the Resource Safety Index (RSI method) becomes clear in the paradigm of the amalgamated (classical) and individual (structural) approaches to the reliability assessment [6-8]. The transition from classical reliability methods to individual (structural) methods is accompanied by the replacement of mathematical-statistical models by models based on failures physics. Reliability of the system is estimated by individual indicators of its elements reliability. Structural reliability methods are used, mainly, for RBI-maintenance. With RCM-maintenance, the methods of classical reliability remain effective.

For a system of $i$ elements and $k$ degradation processes, the general safety index $\beta_{SR}$ is determined through the individual indices $\beta_{dR}$ and the criticality of the element and process $u_{ik}$ as:

$$\beta_{ER} = \lg \left( \sum \frac{u_{ik}}{10^{\beta_{dR}}} \right)$$

(2)

The advanced service policy on technical conditions to which belongs RBI (risk based inspection)-maintenance, does not involve the decommissioning of a subject through a fixed. After a specified period, technical condition check is provided, after which a decision is taken on further operation of the facility. This period, according to the principle of ALARP, depending on the probability and severity of the failure is no less than 1 year [9, 10].

This does not mean that in the period between inspections, one should forget about monitoring the state of the structure. Otherwise, RBI service will turn to preventive maintenance. At RBI, the problem of increasing the degree of exhaustion of a physical resource is solved. However, the required level of security is respected. The evaluation of the technical condition is carried out by checking the diagnostic parameters that react to the action of damaging processes. However, fatigue damage is still difficult to control by direct (physical) methods. In practice, this is done through load monitoring. This indirect method needs regular, not periodic, registration. Organization a similar procedure is difficult. Therefore, the principle of Standardised Load Histories is used, due to which it is possible to refuse continuous monitoring, but to control the accumulation of damages [11, 12].

With the correct application of diagnostic algorithms, the forecast reliability indicators are adequate to the actual ones. The required level of safety is observed. Its decrease, as a rule, is due to the appearance of rare overloads. Then the loading process can be modeled as block, consisting of the main background process with damage per cycle $d_b$ and overloads with damage $d_o$. This approach requires the availability of extended data on fatigue resistance. This article is devoted to their consideration.

3. Models of Fatigue Resistance

From the above description of the RSI method, the requirements for fatigue models follow:

- The ability to quickly find of the lifetime distribution functions (LDF) at different stages of operation;
- The identification of fatigue models should increase the accuracy of the LDF forecast, which is tantamount to increased safety.

In order to quickly determine the damage $d$ through the durability of $N$ as $d = 1 / N$, instead of the traditional S-N curve, the lifetime general equation (LGE) should be used, where by the arguments are the main influence factors - amplitude stress $\sigma_a$, cycle asymmetry coefficient (stress ratio) $R_0$, theoretical coefficient stress concentration $\alpha_c$: 377
\[ \log N = b_0 - m \log \sigma_a - b_1 R - b_2 \alpha_\sigma + b_3 R^2 - b_{4a} \alpha_\sigma^2 + b_{5a} \alpha_\sigma \cdot \log \sigma_a \]  \tag{3}

Where \( b_1, b_2, b_3 \) - sensitivity coefficients to the effect of factors; \( m \) – rate of S-N- curve (slope) or sensitivity to fatigue stress amplitude.

For fixed values of \( R, \alpha_\sigma \), the LGE are transformed into S-N-curves; for fixed values of \( N \) and \( \sigma_a \), analogues of limiting amplitudes are obtained; from the fixed values of \( \sigma_a \) and \( \sigma_m, R, \) the effective stress concentration coefficients are calculated from the function \( N(\alpha) \) (Figure 2). The LGE (3) is presented for the strength criteria of fatigue, taking into account the main factors of influence. With the use of more universal deformation and energy fatigue criteria, the number of members of the LGE will be reduced. The modified Basquin-Manson-Coffin model was successfully applied to the supporting structures [13]:

\[ \varepsilon_a = \frac{1}{4 N^\alpha} \cdot \left( \frac{1}{1-\psi} + \frac{\sigma_{ar}}{E} \cdot \left( \frac{N_a}{N} \right)^{1/m} \right) \]  \tag{4}

Where \( \psi \) is the coefficient of relative constriction; \( \sigma_{ar} \) is the endurance limit based on \( N_a \) cycles; \( E \) is the modulus of elasticity of steel; \( \varepsilon_a \) is the amplitude deformation of the cycle.

The novelty of this equation is that it uses the rate of S-N- curve (slope) \( m \) obtained in the LGE model with high cycle fatigue.

The second basic relationship, combining the results of fatigue tests, is the equation for the dispersion of durability (EDD). It is represented by the linearized function of the lifetime logarithm standard deviation \( S_{\log N} \) from its median value \( \log N \). Its sloping section is described by the equation (Figure 3):

\[ S_{\log N} = B + k_L (\log N - \log N_A) \]  \tag{5}

The parameters of the equation \( B, k_L, \log N_A \) are determined experimentally when obtaining a fatigue reference curve, which precedes the experiments to obtain the LGE. This dependence is necessary for the search the lifetime distribution function, which is involved in the Resource Safety Index assessment.

Figure 2. Transformation of LGE plots

Figure 3. The diagram for assessing the dispersion of durability under strength (\( B_c \)) and deformation (\( B_d \)) criteria
The third model is the accumulated damage function \(a_{ol}^0\):

\[
a_{ol}^0 = a_{ol} + \Delta a_b + \Delta a_R
\]

(6)

Where \(a_{ol} = a_{ol}(X_{ol})\) – basic function of accumulated damage; \(\Delta a_b\) – correction of the base function from the factor \(X_{ol} = d_{ol}/d_b\) (Figure 4). The basic function has no monotonic behavior and can be represented as:

\[
a_{ol} = \log \left[ 10^p X_{ol}^{-m_1} \exp \left( -\frac{X_{ol}}{X_0} \right) + 10^q X_{ol}^{-m_2} \left( 1 - \exp \left( -\frac{X_{ol}}{X_0} \right) \right) \right]
\]

(7)

The parameters of the equation \(p, A, X_0, m_1, m_2\) are partially determined experimentally, partially selected by recommendations. Details of the algorithm for the correction of the accumulated damage, its relationship with models of cracks retarding during overloads, is given in [14].

The combination of three models of fatigue resistance allows not only to locate accurately the LDF, but also to solve the problem of very high cycle fatigue [15, 16]. For this region it is difficult to establish experimentally the limits of endurance. The idea of using block-program tests for this is not new. Due to it there are accelerated fatigue tests. In this aspect, the function \(\Delta a_b\) is actual. Knowing its behavior at \(X_b < 1\), we can estimate the lifetime at very small amplitudes of the baseline process (Figure 4).

**Figure 4. Change of accumulated damage under the influence of the main overload parameters on the areas of low cycle (LCF), high cycle (HCF) and very high cycle (VHCF) fatigue**

**4. Search of the General Safety Index**

The effectiveness of the RSI method demonstrated by the example of high-strength M18 bolts for connecting structural components (Figure 5). Inattention to threaded connections can lead to sad consequences. Fatigue failure of the threaded rods of the turbine cover led to catastrophe on Sayano-Shushenskaya hydroelectric power plant (2009, Russia). Losses are estimated at hundreds of millions of US dollars [17].

Another case of inattention to bolts is associated with a blast furnace (BF) accident at a metallurgical plant in Port Talbot (2001, United Kingdom). The BF superstructure lifted 0.75 m from its supporting framework. Some 200 ton of hot materials were released out, killing three employees and seriously injuring many others. There had been no regular maintenance of the column head bolts. Prior to incident, all bolts, save for those on one column head, had fractured sometime prior. There was therefore less restriction on the shell BF movement during the incident than might have been the case with intact bolts [18].

The examined bolts are a critical part. In the bolts manufacture it undergo a complex process of mechanical and thermal treating. The thread is strengthened by rolling, and the fillet is also improved by surface-plastic deformation. With quality technologies, the destruction of bolts occurs in the thread (1, Figure 5). In case of non-observance of the technological regimes, the bolts do not pass the fatigue test and the destruction is shifted to the threaded edge 2 or to the fillet 3 (Figure 5). Another degradation process is caused by the appearance of fretting fatigue cracks in the surface of the head (4, Figure 5). Their development period is relatively long, but they also cause the bolt to collapse, though it may retain some bearing capacity.

For dangerous places bolt where fractures occurs (1, 2, 3, 4, Figure 5) obtained the fatigue models (Table 1). Fractures
in places 2 and 3 are typical for imperfect bolts. The first model is the LGE (Eq.3) in the form:

$$\log N = b_0 + m \log \Delta F + b_1 R_\sigma$$

(8)

Where $\Delta F$ is the double amplitude (swing) of the force acting on the bolt in kN.

Table 1. Parameters of fatigue models for dangerous bolt places and their criticality (LGE is obtained for twice the amplitude of the forces $\Delta F$ (kN))

<table>
<thead>
<tr>
<th>Dangerous place</th>
<th>LGE</th>
<th>EDD</th>
<th>$u_k$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thread, 1</td>
<td>$b_0$</td>
<td>$m$</td>
<td>$b_1$</td>
</tr>
<tr>
<td>Head surface, 4</td>
<td>30.7</td>
<td>-13.2</td>
<td>-2.5</td>
</tr>
<tr>
<td>Fillets, 2,3</td>
<td>14.5</td>
<td>-4.5</td>
<td>-2.5</td>
</tr>
</tbody>
</table>

Figure 5. The initial safety indices $\beta_{\Sigma P_0}$ of the bolt M18 with the tightening forces $F = 75.6$ kN (F1), $F = 151.2$ kN (F2) and the coefficients of the variation of the double amplitude of the load $\Delta F V = 0.05$ (V1), 0.1 (V2), 0.2 (V3)

The task was to find the cyclic lifetime of $N_R$, which guarantees the safety of $R = PS = 0.98$. For this purpose, according to (2), the value $\beta_{\Sigma P_0} = \log N_R$ is determined (Figure 1). The calculation is performed for a fixed tightening forces $F$ generated in the body of the bolt tension 0.3 and 0.6 of yield strength. Changing the value of external loading $\Delta F$ alters stress ratio $R_\sigma$. Therefore, the use of LGE is convenient.

On the basis of the developed algorithm, the diagrams of the initial RSI $\beta_{\Sigma P_0}$ were obtained (Figure 5). Since during construction, the external load is considered as a variative, this diagram is, in essence, a fatigue curve for the random load at the PS = 0.98.

Given the guaranteed lifetime of the bolt under the several damaging process (Figure 6), $u_k$ criticality index can be considered as a powerful tool for regulating systems reliability. This conclusion follows from the fact that the guaranteed lifetime increases 4-5 times during the transition from the situation $u_k = 1$ to the algorithm with actually calculated $u_k < 1$. That is, in the first situation, the amalgamated safety index is significantly lower than the average $\bar{\beta}_{\Sigma P}$ between the individual indices: $\beta_{\Sigma P} < \bar{\beta}_{\Sigma P}$. In the second situation, the principle $\beta_{\Sigma P} \rightarrow \bar{\beta}_{\Sigma P}$ is formalized. In this case, the method of amalgamation LDF (LDFΣ, Figure 6) gives a very conservative result. Therefore, this method should not be considered universal.
Studies have shown that the factor of variation in external loading of $V_F$ has a more significant effect on the guaranteed lifetime than the tightening force $F$. It should be noted that the increase in the tightening force reduces the fatigue strength to a certain limit. After that, the negative effect of the tightening disappears, because at the root of the first tread there is a local plastic deformation [19]. For large tightening forces $F$, the intensity of falling diagram $\beta_{\Sigma R_0}(\Delta F)$ increases, and for small $F$, this intensity decreases. This can be explained by the fact that at high values of $F$ and small $\Delta F$ there is a high asymmetry of the cycle $R_\sigma$. In such conditions, high-strength steels lose sensitivity to it. With increasing $\Delta F$ and constancy of the force of tightening $F$ the value of $R_\sigma$ decreases.

Therefore, at $R_\sigma < 0.5$, the difference in durability becomes noticeable. Thus, increasing the tightening force is effective.

The diagrams obtained allow 3-5 times longer use of the details than with the forecast by traditional means (Figure 6). Considering the defined guaranteed lifetime of the object under of the system of damaging processes, we can consider the index of criticality $u_{ik}$ as a powerful instrument for the regulation of amalgamated reliability. Such a conclusion follows from the fact that the guaranteed lifetime increases 4-5 times in the transition from the situation $u_{ik} = I$ to the algorithm with the actual calculated $u_{ik} < I$ (Figure 6).

5. Conclusion

The Resource Safety Index is a complex indicator of the technical state that unites damaging processes of various nature. Its application is decisive to the basic requirement of RBI-maintenance: operation of the facility to a pre-failure state at an acceptable level of risk. Three proposed models of fatigue resistance allow to find individual safety indexes of a technical system elements at different stages of the life cycle. Thanks to RSI method forecasting, during RBI-maintenance, parts can be used 3-5 times longer than with traditional methods.

It is proposed to present the results of stationary fatigue tests with two basic models: the lifetime general equation and the lifetime dispersion equation. The first model allows you to abandon laborious tests to determine the endurance limit. The second model makes it possible to establish the parameters of the LDF without resorting to the procedure term-by-term variation of the fatigue curve parameters. As a result, the forecast is more accuracy. The relationship between the parameters of the second model for deformation and strength criteria is shown.

The risk and safety levels are significantly affected, not so much by the designed operating conditions as by deviations from them. That is, extreme conditions, which include overloads. Their influence is taken into account in the third model of fatigue resistance. Based on a generalization of the results of a materials resistance study under cyclic loading with overloads, an algorithm for correcting accumulated damage has been developed. It reflects the nonmonotonic behavior of the accumulated damage $a_0$ under the load of the main background process and the relative overloads level, as well as depending on their stress ratio cycle. The intensity of damage accumulation during loading with overloads depends both on the parameters of the mode, which affect the path of the function $a_0$ and the value of $a_{0\max}$, and on the properties of the material, which determine the value of $a_{0\max}$. The parameters of the loading mode are already set directly in the damage. This ensures the universality of the proposed correcting algorithm.
The algorithm for designing the diagram "general safety index $\beta_{EP} - load parameter (in this case - $AF$)" is a definite alternative to the damage summarization procedure. The latter is relevant in the earlier stages of design, when the uncertainty of the load forces the choice of spectra with wide variability. After the stages of working out of strength, reliability, and even more so at the stage of operation, when the loading process is monitored, its parameter variation is significantly reduced. An opportunity is created without summarizing the damage right away directly to control the exhaustion of the resource.

6. Conflicts of Interest
The authors declare no conflict of interest.

7. References

Particle Swarm Optimization Based Approach for Estimation of Costs and Duration of Construction Projects

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Abstract

Cost and duration estimation is essential for the success of construction projects. The importance of decision making in cost and duration estimation for building design processes points to a need for an estimation tool for both designers and project managers. Particle swarm optimization (PSO), as the tools of soft computing techniques, offer significant potential in this field. This study presents the proposal of an approach to the estimation of construction costs and duration of construction projects, which is based on PSO approach. The general applicability of PSO in the formulated problem with cost and duration estimation is examined. A series of 60 projects collected from constructed government projects were utilized to build the proposed models. Eight input parameters, such as volume of bricks, the volume of concrete, footing type, elevators number, total floors area, area of the ground floor, floors number, and security status are used in building the proposed model. The results displayed that the PSO models can be an alternative approach to evaluate the cost and duration of construction projects. The developed model provides high prediction accuracy, with a low mean (0.97 and 0.99) and CoV (10.87% and 4.94%) values. A comparison of the models’ results indicated that predicting with PSO was importantly more precise.

Keywords: Cost; Duration; Construction Project; Particle Swarm Optimization; Managing Projects; Decision Making.

1. Introduction

The cost and duration prediction is considered an essential issue in construction projects. Underestimation and overestimation of costs may result from the failure of a construction project. The utilize of various approaches in the entire project lifetime should supply information on costs to the contributors to the project and support a complicated decision-making process [1, 2]. Cost and duration evaluating is a vital task for costing and tender preparation for any construction project before they are built. Cost and duration estimation in the early steps of construction projects comprises a considerable doubt. Hence, there is a high request to construct an active approach to minimize uncertainty in cost and duration prediction. One traditional technique for predicting the cost and duration values is the utilize of several specialists. Nevertheless, continuous contact with these specialists is not always an easy choice, producing to improve the alternative method to predict the cost and duration of construction projects. It is preferred to construct the new method according to datasets created from the preceding similar projects. Furthermore, utilizing the traditional technique is difficult and complicated. Hence, utilizing a soft-computing method is a noticeably, more effective way to address nonlinear problems. The best solutions for any system can be defined as the viable solutions with fitness values.

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as well as the values of any other sustainable solutions; these solutions are achieved by selecting values for the set of parameters that satisfy all constraint solutions [3]. Furthermore, optimization approaches are utilized widely in numerous areas, such as engineering and computer science. Study in the optimization area is very active, and new optimization approaches are being developed frequently [4]. The main goal of optimization methods is to find values for a set of parameters that maximize or minimize objective functions that are subject to certain constraints [5]. Through the last decades, numerous researches have been achieved to develop optimization approaches that apply evolutionary programming methods.

A relationship between completed construction cost and the time taken to complete a construction project was first mathematically established by Bromilow (1974) [6]. A regression analysis was utilized by Carr (1979) [7] to organize the duration and cost preparation of industrial buildings. Based on the neural network technique, Wang et al. (2013) [8] proposed a cost estimator model. The learning steps of their neural network were accomplished using a particle swarm optimization (PSO) method. In 2014, a hybrid model PSO-BPNN was proposed by Hong et al. (2014) [9] to assess the cost of construction projects. The PSO technique in the network has optimized the ANN weights. In 2015, Zima (2015) [10] presented a CBR model to predict the construction elements unit price. The CBR method shows a knowledge base that supports the cost prediction at the initial step of a construction project. A hybrid model ANN-ACO and ANN model for determining the amount and cost of construction waste in the early stage of construction were developed by Lee et al. (2016) [11] using "ant colony optimization" ACO algorithm to optimize the ANN weights and ANN model. In 2018, a proposed model for predicting the construction costs of sports areas was presented by Juszczyk et al. (2018) [12]. Hybrid DES-PSO model that includes discrete event simulation (DES) and particle swarm optimization (PSO) algorithms were developed by Hegazy et al. (1994) [13] construction through a set of iterations in networks utilized, that significantly reduces efforts in search optimization scenarios.

The main target of the current research is to propose and investigate models of cost and duration estimating for the construction projects in the initial planning stage using particle swarm optimization (PSO) technique. The proposed PSO models can assist the engineers in making informed decisions in the initial stages of the design steps. With these models, it is probable to acquire a precise estimation, even when suitable information is not obtainable in the initial phases. These approaches encourage a feedback procedure that may support designers to attain the best solution. Moreover, the proposed models considered some category parameters, such as the security status that has been happened in Iraq in the last decade.

2. Research Methodology

Soft-computing methods are utilized to overcome complicated numerical optimization problems as non-linear systems. The current study tries to propose PSO models for predicting the cost and duration of construction projects accurately. The primary purpose of this study is to adopt and propose new models for the duration/cost assessment of construction projects utilizing the PSO algorithm. The proposed models were developed according to numerous, affecting input parameters, as presented below. The definition of the input parameters is listed in Table 1.

\[
\text{Cost/Duration} = f(C,B,EN,FT,AGF,TFA,FN,SS)
\]

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>( C )</td>
<td>The concrete volume: The concrete works comprise lean concrete, screed concrete, foundation, columns, beams, and slabs.</td>
</tr>
<tr>
<td>( B )</td>
<td>The brick volume.</td>
</tr>
<tr>
<td>( EN )</td>
<td>The number of elevators in the buildings.</td>
</tr>
<tr>
<td>( FT )</td>
<td>Types of footing: 1- Raft footing and 2- Separated footing.</td>
</tr>
<tr>
<td>( AGF )</td>
<td>The area of the ground floor.</td>
</tr>
<tr>
<td>( TFA )</td>
<td>The total area of floors.</td>
</tr>
<tr>
<td>( FN )</td>
<td>The floors number.</td>
</tr>
<tr>
<td>( SS )</td>
<td>The security status: 1- Safe, 2-Moderate and 3- Not safe</td>
</tr>
</tbody>
</table>

Optimization is required to produce optimal cost and/or duration values for a construction project. Of three key points must be taken in its progress:

(a) The objective function must be formulated.
(b) A clear approach is required to solve the optimization problem.
(c) The convergence criterion should be specified.

These detailed items will be discussed in the subsequent sections.
2.1. Objective Function of PSO Models

The primary objective of PSO is to optimize the cost and-or duration values and exploration for an optimum set of unknown coefficients, as illustrated in the proposed model section from within the solution space. The actual and forecast values of the duration and cost amount were detected to have minimal differences when using the final form of the optimized model. The proposed models are simulated utilizing MATLAB to optimize the duration and cost amount model for the construction projects. The objective function used in this study is the root mean square error (RMSE). This objective function can be accounted for utilizing the following expression [14-17]:

\[
RMSE = \frac{1}{n} \sum_{i=1}^{n} |y - y'| 
\]  
(2)

Where \(y'\) refers to the forecasted value, \(y\) refers to the actual value, and \(n\) denotes to the number of dataset samples.

2.2. Optimization Method of PSO Models

As a result of its global convergence ability, easy implementation, and adoption, PSO is considered one of the best optimization approaches. PSO is an evolutionary computation approach developed by Eberhart et al. (2001) [18], which was inspired by the social behavior of bird (particle) flocking. The PSO algorithm is generally accepted and used in solving different optimization problems. During the entire search process, the position and velocity of each particle can be updated according to Equations 3 and 4.

\[
V_i(t + 1) = wV_i(t) + c_1Rand(\cdot)pbest_i - X_i(t) + c_2Rand(\cdot)gbest_i - X_i(t) 
\]  
(3)

\[
X_i(t + 1) = X_i(t) + V_i(t + 1) 
\]  
(4)

Where \(V_i\) and \(X_i\) are the velocity and position of the particles, respectively; \(Rand(\cdot)\) and \(Rand(\cdot)\) are random numbers that are uniformly distributed between 0 and 1; \(pbest\) denotes the best position of each particle in space, and \(gbest\) represents the globally best position of all the particles. Acceleration coefficients \(c_1\) and \(c_2\) describe the ‘trust’ settings that mention the degree of confidence in the optimal solution found by an individual particle (\(c_1\)-cognitive parameter) and by the whole swarm (\(c_2\)-social parameter). The term \(w\) in Equation 3 refers to the inertial weight that was presented to improve the convergence of the iteration procedure. This weight is a scaling factor utilized to control the search capabilities of the swarm, which scales the current velocity value that affects the updated velocity vector. Later, Shi and Eberhart (1998) [19] developed the original PSO algorithm by adding the inertial weight; thus, this weight was not a portion of the original one.

![Figure 1. Pictorial view of particle behavior showing position and velocity update [20]](image-url)
Figure 1 displays the updated particle position and velocity of the 2D parameter space. The first vector refers to the momentum velocity of the particles in the previous stage. The second vector refers to the particle memory components that get the best position as a result of iteration. This speed component attracted the particle to the best position in the solution space. So, the last vector is called a social component or swarm. The particle in this component is attracted to the best position in the swarm [20].

2.3. Convergence Criteria

Convergence criteria must be applied to end the process of optimization during the repeated search [21, 22]. The maximum number of iterations and minimum error requirements are the convergence criteria adopted in the PSO algorithm. The complexity of the optimization problem determines the maximum number of iterations. Previous knowledge of the optimal global error value determines the minimum error of the algorithm, which is possible to test or adjust the algorithm in mathematical problems when optimization is known a priori. Table 2 lists the main PSO parameters. Table 3 illustrate the convergence parameters of the PSO utilized in the current study [23].

<table>
<thead>
<tr>
<th>Table 2. Main PSO parameters [23]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
</tr>
<tr>
<td>Number of particles, ( NP )</td>
</tr>
<tr>
<td>The dimension of particles, ( n )</td>
</tr>
<tr>
<td>Inertia weight, ( w )</td>
</tr>
<tr>
<td>Vectors containing the lower and upper bounds of the ( n ) design variables, respectively, ( x^L ), ( x^U )</td>
</tr>
<tr>
<td>Cognitive and social parameters, ( c_1 ) and ( c_2 )</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 3. PSO convergence parameters [23]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Description</td>
</tr>
<tr>
<td>Maximum number of iterations (( t_{\text{max}} )) for the termination criterion</td>
</tr>
<tr>
<td>Number of iterations (( k_f )) for which the relative improvement of the objective function satisfies the convergence check</td>
</tr>
<tr>
<td>Minimum relative improvement (( f_m )) of the value of the objective function</td>
</tr>
</tbody>
</table>

2.4. Proposed PSO Model

Figure 2 displays the flow chart of the proposed PSO model utilized in the current study. The correct choice of the PSO parameters has a remarkable effect on the performance of the algorithm. These parameters are namely: swarm size, coefficient of inertia, and acceleration coefficient. The neighborhood size is also needed for the local best algorithm.
The optimization procedure typically uses a gradient-based algorithm appropriate for local exploration. Consequently, to be successful, the optimization procedure needs an initial point gotten from a global exploration. A strong training process requires both the initialization and optimization procedures. The following highlights how the PSO algorithm can be implemented to search for the optimum duration and cost amount of the construction projects.

- Create a swarm initialization by assigning a random location for each particle in the hyperspace problem.
- Evaluate the objective function of the proposed model for each particle.
- Compare the objective function value of each particle with pbest. If the current value is better than the pbest value, this value is set as pbest, and the position of the current particle, Xi, is set to pbest.
- Identify particles with the best objective function value. The value of its target function is determined to be gbest, and its location is gbest.
- Update the velocity and the position of all particles based on Equations 3 and 4.
- Repeat steps 2-5 until the convergence criteria are met (the maximum number of iterations or a sufficient objective function value is obtained).

The proposed model was formulated utilizing MATLAB software to optimize the cost amount and duration models of construction projects. The proposed models to be optimized are as follows:

\[
\text{Cost} = F_1 + F_2 \cdot SS + F_3 \cdot FN + F_4 \cdot C + F_5 \cdot B + F_6 \cdot EN + F_7 \cdot TA + F_8 \cdot AG + F_9 \cdot FT
\] (5)
\[ \text{Duration} = K_1 + K_2 \cdot SS + K_3 \cdot FN + K_4 \cdot C + K_5 \cdot B + K_6 \cdot EN + K_7 \cdot TA + K_8 \cdot AG + K_9 \cdot FT \] (6)

Where \(F_1\) to \(F_9\) and \(K_1\) to \(K_9\) are the unknown coefficients.

The main goal of utilizing PSO to optimize the cost amount and duration models is to examine for an optimum set of unknown coefficients. Hence, the difference between the actual cost amount of construction projects and that predicted utilizing the final form of the optimized expressions is minimal.

### 2.5. Description of Dataset

A total of 60 construction projects constructed by government contractors between 2008 and 2016 from different places in Iraq were collected. The selected projects (samples) represent about 80% of the projects implemented in Iraq in terms of implementation method, materials used, and architectural style. Eight input variables \((C, B, EN, FT, AGF, TFA, FN, SS)\) and two output variables (cost amount or duration), as displayed in Table 4.

Models inferred using optimization tools have the capability to estimate within the data range obtainable and are applied for additional development. Thus, the size of the dataset utilized for the modeling process is essential, as it impacts the accuracy of the final models. The behavior of any model modified using this data is influenced by the sample size and its variable distributions. Therefore, the data is graphically illustrated in Figure 3 as histograms. Figure 3 depicts the statistics of the samples utilized in constructing the proposed model.

For high accuracy, the ratio of the number of dataset records to the number of input parameters should not be less than three, as proposed by Frank and Todeschini (1994) [24], and they recommended to be higher than five. For the present case study, this ratio was \(60/8 = 7.5\), which exceed the recommended criteria. From the 60 samples (projects), 48 samples (80 %) were considered for building the proposed models, while 12 samples (20 %) were utilized to validate the proposed models. The descriptive statistics of the dataset utilized in this study are given in Table 5.

### Table 4. Input and Output Parameters

<table>
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<tr>
<th>Item</th>
<th>Project ID</th>
<th>SS</th>
<th>FN</th>
<th>C</th>
<th>B</th>
<th>EN</th>
<th>FT</th>
<th>AGF</th>
<th>GFA</th>
<th>Cost</th>
<th>Duration</th>
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<td>665.824</td>
<td>186</td>
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<td>1370</td>
<td>2400</td>
<td>1715.766</td>
<td>287</td>
<td></td>
</tr>
<tr>
<td>Directorate of AL-Awqaf /AL-Anbar</td>
<td>3</td>
<td>5</td>
<td>1828</td>
<td>2116</td>
<td>3</td>
<td>2</td>
<td>1040</td>
<td>4500</td>
<td>3445.0983</td>
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<tr>
<td>Internal departments for students of University of Karbala/Karbala</td>
<td>2</td>
<td>4</td>
<td>1605</td>
<td>1709</td>
<td>1</td>
<td>2</td>
<td>1040</td>
<td>3600</td>
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<td>Fatima Al - Zahra Secondary School/Waset</td>
<td>2</td>
<td>2</td>
<td>241</td>
<td>243</td>
<td>0</td>
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<td>322</td>
<td>494</td>
<td>307.585</td>
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<td></td>
</tr>
<tr>
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<td>1</td>
<td>5</td>
<td>1279</td>
<td>1140</td>
<td>1</td>
<td>2</td>
<td>593</td>
<td>2470</td>
<td>1554.1911</td>
<td>395</td>
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<tr>
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<td>1</td>
<td>4</td>
<td>1479</td>
<td>1360</td>
<td>2</td>
<td>2</td>
<td>865</td>
<td>2964</td>
<td>1923.39945</td>
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<td>2244</td>
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<td>2</td>
<td>1394</td>
<td>4888</td>
<td>3695.29965</td>
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<tr>
<td>Internal sections for officers and security associates/Ministry of Interior / Baghdad</td>
<td>1</td>
<td>2</td>
<td>1687</td>
<td>1701</td>
<td>0</td>
<td>1</td>
<td>1952</td>
<td>3458</td>
<td>1956.013</td>
<td>287</td>
<td></td>
</tr>
<tr>
<td>Abdullah bin Rawahah School/Baghdad</td>
<td>1</td>
<td>2</td>
<td>326</td>
<td>230</td>
<td>0</td>
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<td>387</td>
<td>612</td>
<td>335.235</td>
<td>160</td>
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</tr>
<tr>
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<td>4</td>
<td>1087</td>
<td>689</td>
<td>1</td>
<td>2</td>
<td>589</td>
<td>1960</td>
<td>1214.8521</td>
<td>321</td>
<td></td>
</tr>
<tr>
<td>Administrative building of the Ministry of Interior/Kirkuk</td>
<td>2</td>
<td>5</td>
<td>1321</td>
<td>850</td>
<td>1</td>
<td>2</td>
<td>589</td>
<td>2450</td>
<td>1582.87395</td>
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</tr>
<tr>
<td>Administrative building of the Ministry of Municipalities/ Salah Eddin</td>
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<td>4</td>
<td>1848</td>
<td>1396</td>
<td>2</td>
<td>2</td>
<td>966</td>
<td>3332</td>
<td>2601.17445</td>
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<tr>
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<td>3</td>
<td>6</td>
<td>2255</td>
<td>1433</td>
<td>2</td>
<td>2</td>
<td>829</td>
<td>4248</td>
<td>3177.9678</td>
<td>621</td>
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</tr>
<tr>
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<td>6</td>
<td>2217</td>
<td>1305</td>
<td>2</td>
<td>2</td>
<td>754</td>
<td>3840</td>
<td>2322.09285</td>
<td>502</td>
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</tr>
<tr>
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<td>2</td>
<td>5</td>
<td>2947</td>
<td>1950</td>
<td>2</td>
<td>2</td>
<td>1106</td>
<td>4800</td>
<td>3250.77585</td>
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<td></td>
</tr>
<tr>
<td>Administrative building of the Ministry of Municipalities / Ninava</td>
<td>3</td>
<td>5</td>
<td>5652</td>
<td>3315</td>
<td>4</td>
<td>2</td>
<td>1849</td>
<td>8175</td>
<td>6263.49045</td>
<td>746</td>
<td></td>
</tr>
<tr>
<td>Administrative building of the Ministry of Municipalities/ Wasit</td>
<td>2</td>
<td>4</td>
<td>720</td>
<td>480</td>
<td>1</td>
<td>2</td>
<td>402</td>
<td>1280</td>
<td>899.02575</td>
<td>323</td>
<td></td>
</tr>
<tr>
<td>Administrative building of the Ministry of Interior/Baghdad</td>
<td>1</td>
<td>5</td>
<td>1350</td>
<td>910</td>
<td>1</td>
<td>2</td>
<td>578</td>
<td>2400</td>
<td>1421.4354</td>
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</tr>
</tbody>
</table>
Figure 3. Histograms of independent input variables
Table 5. Descriptive statistics of the variables used in the model development

<table>
<thead>
<tr>
<th>Description</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>Range</th>
<th>Min.</th>
<th>Max.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Input</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>SS</td>
<td>1.88</td>
<td>0.885</td>
<td>2</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>FN</td>
<td>3.35</td>
<td>1.686</td>
<td>6</td>
<td>1</td>
<td>7</td>
</tr>
<tr>
<td>C</td>
<td>1515.83</td>
<td>1773.455</td>
<td>8262.5</td>
<td>20.5</td>
<td>8283</td>
</tr>
<tr>
<td>B</td>
<td>955.62</td>
<td>669.247</td>
<td>3287</td>
<td>28</td>
<td>3315</td>
</tr>
<tr>
<td>NOE</td>
<td>1.167</td>
<td>1.2645</td>
<td>4</td>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>FT</td>
<td>1.5</td>
<td>0.504</td>
<td>1</td>
<td>1</td>
<td>2</td>
</tr>
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<td>AGF</td>
<td>941.27</td>
<td>631.60</td>
<td>4334</td>
<td>116</td>
<td>4450</td>
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<td>GFA</td>
<td>2869.47</td>
<td>2419.052</td>
<td>9740</td>
<td>60</td>
<td>9800</td>
</tr>
<tr>
<td><strong>Output</strong></td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total Cost</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3. Results and Discussion

The PSO technique was utilized to optimize the construction projects cost and/or duration amount. Models have been proposed to examine the influences of swarm size on the outcomes. The main job of the objective function in a PSO approach is to reduce the difference between the predicted and actual cost and/or duration amount. PSO offers models that can assess the cost and/or duration and finding results as close as possible to the measured results. The PSO technique updates its process until either a proper global best (gbest) or the maximum epochs (iterations) is achieved, as presented in the methodology. Table 6 shows the parameters used in the PSO model.

Table 6. Parameters of the PSO

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Swarm size</td>
<td>10, 20, 30, 40, and 50</td>
</tr>
<tr>
<td>Target error</td>
<td>1e-05</td>
</tr>
<tr>
<td>Iteration</td>
<td>10000</td>
</tr>
<tr>
<td>C₁</td>
<td>1.495</td>
</tr>
<tr>
<td>C₂</td>
<td>1.495</td>
</tr>
<tr>
<td>w</td>
<td>0.7</td>
</tr>
</tbody>
</table>

Statistical methods, namely: the coefficient of variation (CoV), correlation coefficient (R), and Bland–Altman (2007) [25] analysis, were used in this study to evaluate and examine the ability of the proposed models. The root mean square error (RMSE) was used as an objective function to choose unknown coefficients. Additionally, five swarm sizes (10, 20, 30, 40, and 50) were used and evaluated. In this study, the iterations number fixed to 1000 because of the differences in the objective functions are stabilized after 700 iterations, as shown in Figure 4 for both cost and duration models. Numerous swarm sizes were tested to evaluate which swarms could minimize the error.

![Cost model](image1)

![Duration model](image2)

Figure 4. Objective function (RMSE) versus iteration.
The Bland–Altman (2007) [25] approach was applied to check the agreement between the actual (A) and predicted (B) cost/duration. The difference between the actual and estimated cost/duration (A−B) is plotted against their mean value ((A+B)/2), as presented in Figure 5. The Bland–Altman plot exposes the variances in the results and presentations the systematic and random variances. The average value (m) of +1.96 and standard deviation (SD) of −1.96 are displayed in the diagram. These values are called the “limits of agreement”. Such limits of agreement specify whether the limits are large or small concerning the overall values, where values outside these ranges show low accuracy.

a) Cost model
Figure 5. Bland–Altman plot of the relationship between actual and predicted cost and duration values for 50 swarms

As previously determined, Bland-Altman analysis predicts the level of difference. Monitoring scattered values can help to find agreement between actual and predicted values. As shown in Figure 5, a reasonable agreement between the test methods was presented. This figure shows that the data is distributed within the limits of the agreement, indicating the appropriate accuracy of the proposed models. Figure 4 illustrates that 50 swarms provided a better solution for the PSO because, they accomplished the minimum objective functions with 95.83% and 97.92% accuracy, for cost and duration, respectively. With respect to the other swarm sizes, 10 swarms produced significant errors. The results show that the 50 swarms displayed a higher accuracy for the actual values for both models, namely: cost and duration.
According to the CoV and $R$ values, the proposed models accomplish minimum error, as presented in Table 7. The best solution for the PSO algorithm because it accomplishes a minimum coefficient of variation, CoV, and maximum value of correlation coefficient ($R$), as presented in Table 7. Smith (1986) [26] recommended a rational hypothesis to judge the performance of the model by the following criteria:

- If a model gives $|R| > 0.8$, a strong correlation occurs between the forecast and actual values;
- If a model gives $0.2 < |R| < 0.8$, a good correlation occurs between the forecast and actual values;
- If a model gives $|R| < 0.2$, a weak correlation occurs between the forecast and actual values.

Figure 6 displays that the proposed PSO models had an adequate R-values (0.9441 and 0.9940 for cost and duration) and assessed the target values with adequate accuracy. Moreover, the coefficients ($F_1$ to $F_9$) and ($K_1$ to $K_9$) obtained from the optimization results will be substituted in Eqs. 5 and 6 of the proposed models, as presented in the following final expressions.

Cost = $-611.346 + 156.283 \cdot SS + 83.98399 \cdot FN + 0.610803 \cdot C + 0.567313 \cdot B + 92.57055 \cdot EN − 0.16582 \cdot TA + 0.610803 \cdot AG + 0.567313 \cdot FT$

Duration = $2.903068 + 25.6335 \cdot SS + 54.38925 \cdot FN − 0.00249 \cdot C + 0.009338 \cdot B + 13.94593 \cdot EN + 0.036325 \cdot TA + 0.00249 \cdot AG + 0.009338 \cdot FT$

Table 7. Factors used in the PSO-Cost and PSO-duration models setting

<table>
<thead>
<tr>
<th>Factor</th>
<th>PSO-Cost 50 Swarm</th>
<th>Factor</th>
<th>PSO-Duration 50 Swarm</th>
</tr>
</thead>
<tbody>
<tr>
<td>F 1</td>
<td>-611.346</td>
<td>K 1</td>
<td>2.903068</td>
</tr>
<tr>
<td>F 2</td>
<td>156.283</td>
<td>K 2</td>
<td>25.6335</td>
</tr>
<tr>
<td>F 3</td>
<td>83.98399</td>
<td>K 3</td>
<td>54.38925</td>
</tr>
<tr>
<td>F 4</td>
<td>0.610803</td>
<td>K 4</td>
<td>-0.00249</td>
</tr>
<tr>
<td>F 5</td>
<td>0.567313</td>
<td>K 5</td>
<td>0.009338</td>
</tr>
<tr>
<td>F 6</td>
<td>92.57055</td>
<td>K 6</td>
<td>13.94593</td>
</tr>
<tr>
<td>F 7</td>
<td>-0.16582</td>
<td>K 7</td>
<td>0.036325</td>
</tr>
<tr>
<td>F 8</td>
<td>0.610803</td>
<td>K 8</td>
<td>-0.00249</td>
</tr>
<tr>
<td>F 9</td>
<td>0.567313</td>
<td>K 9</td>
<td>0.009338</td>
</tr>
<tr>
<td>M</td>
<td>0.988</td>
<td></td>
<td>0.981</td>
</tr>
<tr>
<td>SD</td>
<td>0.174</td>
<td></td>
<td>0.058</td>
</tr>
<tr>
<td>COV %</td>
<td>17.6</td>
<td></td>
<td>9.0</td>
</tr>
<tr>
<td>R</td>
<td>0.9441</td>
<td></td>
<td>0.9940</td>
</tr>
</tbody>
</table>

Figure 6. Predicted vs. actual cost and duration values using the proposed model
4. The Validity of the Proposed Models

A dataset comprising of 12 construction projects (20% of the total dataset) was utilized to examine and validate the proposed models. These samples were not utilized in the construction stage of the proposed models. Table 8 shows that the cost and duration assessed by the proposed models are reliable and consistent based on the results. The results recorded values of the mean close to 1.0 (0.97 and 0.99 for cost and duration); this reflected the accuracy of the proposed model, as presented in Table 8.

Pimentel-Gomes (2000) [27] specified that the value of a CoV reflects the accuracy of the relationship between the inputs and the output, where CoV values of less than 10%, 20–30%, and above 30% mean high accuracy, low accuracy, and low precision, respectively. For the proposed model, the COVs for cost and duration models were 10.86% and 4.93%, representing high accuracy. Moreover, the $R$-values of 0.9914 and 0.9940 (as presented in Table 8) reflect a good agreement between the actual and forecast cost and duration values. It can be stated based on these results that the proposed models efficiently assess the cost and duration of the construction projects.

Table 8. Actual database and values predicted using the PSO model

<table>
<thead>
<tr>
<th>Item</th>
<th>Project ID</th>
<th>Cost actual</th>
<th>Duration actual</th>
<th>PSO-Cost</th>
<th>Cost Actual/predicted</th>
<th>PSO Duration</th>
<th>Duration Actual/predicted</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Event hall of the Ebad al-Rahman mosque / Yusufiya / Baghdad</td>
<td>177.765</td>
<td>95</td>
<td>207.6</td>
<td>0.86</td>
<td>94.2</td>
<td>1.01</td>
</tr>
<tr>
<td>2</td>
<td>School of Habib Ben Khedi/ Nineveh</td>
<td>1058.018</td>
<td>247</td>
<td>1185.2</td>
<td>0.89</td>
<td>251.3</td>
<td>0.98</td>
</tr>
<tr>
<td>3</td>
<td>Residential Units for Employees/ Nukhayb / Anbar</td>
<td>237.018</td>
<td>177</td>
<td>254.3</td>
<td>0.93</td>
<td>201.6</td>
<td>0.88</td>
</tr>
<tr>
<td>4</td>
<td>Directorate of AL-Awqaf/Kut/ Wasit</td>
<td>768.365</td>
<td>189</td>
<td>743.9</td>
<td>1.03</td>
<td>183.8</td>
<td>1.03</td>
</tr>
<tr>
<td>5</td>
<td>Expanding the building of the Faculty of Engineering/University of Baghdad / Baghdad</td>
<td>907.072</td>
<td>204</td>
<td>1022.9</td>
<td>0.89</td>
<td>200.6</td>
<td>1.02</td>
</tr>
<tr>
<td>6</td>
<td>Administrative building/Council of Governors AL-Anbar /AL-Anbar</td>
<td>2454.04215</td>
<td>456</td>
<td>2238.2</td>
<td>1.10</td>
<td>452.0</td>
<td>1.01</td>
</tr>
<tr>
<td>7</td>
<td>Administrative building of Babylon University/Babylon</td>
<td>2254.33425</td>
<td>439</td>
<td>2015.1</td>
<td>1.12</td>
<td>461.0</td>
<td>0.95</td>
</tr>
<tr>
<td>8</td>
<td>Administrative building for the Ministry of Labor and Social Affair /Waset</td>
<td>1495.8027</td>
<td>422</td>
<td>1493.6</td>
<td>1.01</td>
<td>424.6</td>
<td>0.99</td>
</tr>
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<td>9</td>
<td>Administrative building of the Ministry of oil/ Nineveh</td>
<td>1715.766</td>
<td>287</td>
<td>1955.4</td>
<td>0.88</td>
<td>294.9</td>
<td>0.97</td>
</tr>
<tr>
<td>10</td>
<td>Directorate of AL-Awqaf / Baghdad</td>
<td>1554.1911</td>
<td>395</td>
<td>1591.8</td>
<td>0.98</td>
<td>410.2</td>
<td>0.96</td>
</tr>
<tr>
<td>11</td>
<td>Internal sections for officers and security associates/Ministry of Interior / Baghdad</td>
<td>1956.013</td>
<td>287</td>
<td>2327.8</td>
<td>0.84</td>
<td>269.8</td>
<td>1.06</td>
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<tr>
<td>12</td>
<td>Administrative Building of the Ministry of Municipalities/AL-Anbar</td>
<td>3177.9678</td>
<td>621</td>
<td>2806</td>
<td>1.13</td>
<td>594.1</td>
<td>1.05</td>
</tr>
</tbody>
</table>

| M    | 0.97 | 0.99 |
| SD   | 0.105 | 0.049 |
| CoV %| 10.86 | 4.93 |
| R    | 0.9914 | 0.9940 |

The criteria suggested by Golbraikh et al. (2002) [28] were checked for the external verification of the proposed models on the testing datasets. It seems that at least one slope of regression lines ($k$ or $k'$) through the origin should be close to 1.0. Roy and Roy (2008) [29] introduced a confirmative indicator of the external predictability of models ($R_m$). For $R_m > 0.5$, the condition is satisfied. The squared correlation coefficient (through the origin) between predicted and experimental values ($R_0^2$) should be close to 1.

The considered validation criteria and the relevant results obtained by the model are presented in Table 9. As can be seen, the proposed models satisfy the required conditions. The external validation criteria result for the models are presented in Table 9.
Table 9. Statistical parameters of the PSO models for external validation

<table>
<thead>
<tr>
<th>Item</th>
<th>Formula</th>
<th>Condition</th>
<th>Cost model</th>
<th>Duration model</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$R = \frac{\sum_{i=1}^{n}(EA_i - \bar{EA}) (EE_i - \bar{EE})}{\sqrt{\sum_{i=1}^{n}(EA_i - \bar{EA})^2 \sum_{i=1}^{n}(EE_i - \bar{EE})^2}}$</td>
<td>$R &gt; 0.8$</td>
<td>0.9914</td>
<td>0.9940</td>
</tr>
<tr>
<td>2</td>
<td>$k = \frac{\sum_{i=1}^{n}(EA_i \times EE_i)}{EA_i^2}$</td>
<td>$0.85 &lt; k &lt; 1.15$</td>
<td>1.0592</td>
<td>1.0340</td>
</tr>
<tr>
<td>3</td>
<td>$k1 = \frac{\sum_{i=1}^{n}(EA_i \times EE_i)}{EE_i^2}$</td>
<td>$0.85 &lt; k1 &lt; 1.15$</td>
<td>0.9441</td>
<td>0.9671</td>
</tr>
<tr>
<td>4</td>
<td>$R_m = R^2 \times \left(1 - \sqrt{R^2 - R_o^2}\right)$</td>
<td>$R_m &gt; 0.5$</td>
<td>0.8690</td>
<td>0.7858</td>
</tr>
</tbody>
</table>

where $R_o^2 = 1 - \frac{\sum_{i=1}^{n}(EE_i - EA_o)^2}{(EE_i - EE_i)^2}$, $EA_o = k \times EE_i$

The cost and duration values estimation achieved by the proposed models are illustrated in Figures 7 and 8. The models have acceptable estimation accuracy when the ratio of the actual to estimated values is close to one. As can be presented from Figure 7, the ratio distribution of the actual to estimate values for the proposed PSO model in duration have better estimation accuracy than the PSO model in cost.

For further statistical analysis for the mentioned models, a comparison between the actual and assessed cost and duration values has been illustrated in Figure 8. This Figure shows that the proposed PSO model in the duration is closer to the actual duration of projects than the proposed PSO model in cost.

Figure 7. Comparison between the predicted and actual cost and duration amount using the PSO models

Figure 8. Actual versus predicted cost and duration amount utilizing the PSO models

5. Screening and Parametric Analyses

After constructing the proposed model, various phases were considered: (i) deriving the final models based on collected datasets; (ii) computing several external validation criteria to verify the models; and (iii) conducting a parametric study based on engineering principles and the physics of the problem. The first two steps are purely statistical;
however, the third step is based on engineering principles and should be performed by an engineer who understands the problem being modeled. The first and second steps were achieved here for this type of problem. Therefore, for further verification of the developed model of cost and duration value, a parametric analysis was performed.

This study primarily seeks to assess the effect of individual parameters on cost and duration values. Figure 9 demonstrates the forecast values of the cos and duration accomplished by the proposed models as a function of each parameter. Figure 9 (a) and (b) show the proposed models as a function of the \((C, B, EN, FT, AGF, TFA, FN, SS)\) parameters. Figure 9 (a) and (b) display that increases in the amounts of \(C, B, EN, FT, AGF, TFA, FN, SS\) up to a certain level lead to increases in the cost and duration values, indicating that the proposed models can be utilized as a guide to choose the suitable parameters correctly. Moreover, Figure 9 displays that the parameters SS, C, and GFA are the most effect parameters on the cost and duration values.

Figure 9 Parametric analysis of the cost and duration values using the proposed models
6. Conclusions and Recommendations

The main objective of this study was to develop mathematical models that will be applied to forecast the cost and duration of the construction projects. In this study, sixty construction projects were utilized to build the proposed models at early-stage design. The main conclusions are drawn according to the models’ outcomes, as follows:

- The contractors can utilize the proposed model to assess the construction cost and/or duration, and compare them with that specified by the client at the bid phase, to know if the cost and/or duration will be reasonable for the given project and its budget. This modeling technique based on historical datasets collected from existing projects. Thus, it is more practical, consistent, and reliable than currently utilized subjective methods based on intuitive assessments by designers.

- The statistical analysis demonstrates that the CoV, mean, and $R$ display good accuracy and reliability for the predicted values. With low mean (0.97 and 0.99) and CoV (10.87% and 4.94%) values, the proposed PSO models (for both cost and duration) provide a proper assessment of the construction projects. Hence, this model can be utilized as a design indicator of cost and duration estimations at the early-stage design.

- The outcomes display that the PSO technique is proper for evaluating project management problems and can be utilized as a useful tool to search the optimal solutions with differs parameters.

- The proposed model supplies a guide for choosing the suitable parameters that influence the cost and duration parameters, such as security status, total area, area of the ground floor, floors number, the brick and concrete volume, and elevators number.

In this study, the dataset for only sixty construction projects was utilized to build the model. Nevertheless, more case studies with similar kinds of projects will supply more consistent results.

- Further construction projects should be conducted to examine and modify the proposed model and to investigate a wide range of parameters.

- Future research could be considered to build a model for cost and/or duration estimation for green buildings.

7. Conflicts of Interest

The authors declare no conflict of interest.

8. References


Cementitious, Pozzolanic And Filler Materials For DSM Binders

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Abstract

Deep Soil Mix (DSM) is a proven method of ground improvement for deeper underlying soft soil layers which are otherwise impractical to reach using conventional shallow soil stabilization and replacement methods. The predominant binder materials used are Ordinary Portland cement (OPC) and Lime (CaO) but negative effects to the environment from manufacture and increasing construction cost have prompted research into alternative materials. This review identifies pozzolans and filler materials as possible supplements or partial substitutes for better results. The DSM method and binder reaction processes during treated soil strength development are outlined and effectiveness of different pozzolans (Fly Ash, Silica Fume, Ground Granulated Blast Furnace Slag, Rice Husk Ash, Kaolin, and Metakaolin) and filler materials (e.g. fine sand) discussed together with their influence factors. With many pozzolans, a clear optimum dosage is observed where improved strength peaks. Aluminosilicate pozzolans perform better over siliceous pozzolans with Metakaolin (MK) identified as the most effective pozzolan for enhancing compressive strength. Up to date research results on these materials are compiled. MK blended cements are readily available and can be readily applied for initial field tests. Treated soil strength may be regulated with addition of filler materials to further reduce reliance on cement.

Keywords: Ground Improvement; Lime; Ordinary Portland Cement; Pozzolans; Deep Soil Mix.

1. Introduction

Construction in soft soil conditions require either wide footprint or deep foundations to support the overlying structure without soil shear failure or excessive settlement. Highly developed areas for infrastructure, residential, commercial and industry often encounter soft soil conditions (peat / clay / silt in river deltas, flood plains and alluvial plains etc.).

A viable alternative engineering solution is through ground improvement. The properties of the underlying soil are improved to satisfactorily support the imposed bearing pressure from the structure above. The soil properties can be enhanced by 1) full or partial replacement with stronger materials – e.g. geo-textiles, fibrous materials, etc.; 2) adding binder materials that will react with the soil to strengthen it – e.g. soil stabilization, soil reinforcement, etc.; or 3) modifying the existing consistency of the soil – e.g. pre-loading, electro-kinetic stabilization, etc.

The Deep Soil Mix (DSM) method applies soil stabilization principles, which comprises the addition and mixing of binder materials with the soil as stabilizing agents and other filler materials in the form of columns into the soil stratum. Presently, the pre-dominant binder materials utilized are cement and lime. These are classified as traditional cementitious / hydraulic binder materials.
However, industrial production of cement releases significant carbon dioxide (CO$_2$) (about 7% global man-made CO$_2$ emissions [1]) to the atmosphere which contributes to the climate change effect. Likewise, CO$_2$ is also a by-product when producing quicklime (CaO). Production of both materials is also energy intensive. Hence, there is incentive to research new alternative replacement materials that offer similar or better ground improvement performance but with less environmental, energy impact and financial cost. Pozzolanic and filler materials have been applied successfully to improve concrete properties and researched for soil stabilization. This paper briefly explains the reaction processes and discusses their applicability to supplement or improve performance of traditional binders in the case of DSM.

2. Deep Soil Mix (DSM)

In DSM, specialized equipment auger to depth, and inject binder material / filler material which reacts with the existing soil to form columns of improved soil. The primary objective of deep mixing is to produce a stabilized soil mass consisting of a group of soil-stabilized columns. In contrast, conventional soil stabilization involves applying and mixing stabilizing agents at shallow depth or excavated layer of soil. DSM has been applied in soft ground conditions such as marine clay, alluvial deposits, organic soils and peat. The deep mixing method has been developed and practiced primarily in Japan and Scandinavian countries since the 1980s as well as in the USA and central Europe since the 1990s.

Column installation in DSM may either be 1) Wet mixed (WDSM) or 2) Dry mixed (DDSM). For a wet mixing method, the binding agent is turned into wet slurry form and pressure injected into the soil through nozzles located at the end of a specialized soil auger. Column depths of 45m with 0.5 to 0.9m diameters have been achieved with column compressive strengths ranging from 1.5 to 4 MPa [2]. For the dry mixing method (considered more economical than WDSM), the dry binder is injected (using compressed air) into the soil at depth and thoroughly mixed with moist soil. The soil is pre-mixed using a specialized tool during the downward penetration of the auger until the targeted depth is reached. As the mixing tool is withdrawn, the dry binder is injected and mixed with the soil – forming a moist soil mix column. Typical column diameters range from 0.6 to 0.8m with depths of 25-30m. Design compressive strength is from 150 to 500 kPa [2].

The effectiveness and selection of which DSM installation method is dependent on moisture content of the soil layer. Generally, DSDM techniques achieve less strength for the same soil type than WDSM. WDSM is suitable for soft clays, silts, fine-grained sands with lower water content, and multiple interbedded soft and stiff/dense soil layers. DSDM is more appropriate for soft soils with high moisture content, organic soils and sludges [3].

<table>
<thead>
<tr>
<th>Research</th>
<th>Location</th>
<th>Method</th>
<th>Soil</th>
<th>Binder (kg/m$^3$)</th>
<th>Cement type</th>
<th>Actual UCS (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[4] Japan</td>
<td>DDSM</td>
<td>Clay</td>
<td>130 - 290</td>
<td>Blast Furnace Slag Cement Type B</td>
<td>0.2 – 0.5 (28 d) 1.8 – 4.2 (17 yrs.)</td>
<td></td>
</tr>
<tr>
<td>[5] S. Korea</td>
<td>WDSM</td>
<td>Clay</td>
<td>270</td>
<td>GGBS cement</td>
<td>1.1 @ 2m depth – 5.1 @ 15m depth (28 d)</td>
<td></td>
</tr>
<tr>
<td>[6] USA</td>
<td>WDSM</td>
<td>Residual</td>
<td>Not stated</td>
<td>Not stated</td>
<td>0.69 design / 1.5 avg. (28 d)</td>
<td></td>
</tr>
<tr>
<td>[7] Singapore</td>
<td>WDSM</td>
<td>Clay</td>
<td>280</td>
<td>OPC</td>
<td>1.7 mean</td>
<td></td>
</tr>
<tr>
<td>[8] Vietnam</td>
<td>WDSM</td>
<td>Clay</td>
<td>200 – 240</td>
<td>Tower (60%) slag cement Stable soil cement</td>
<td>2 @ 3m depth – 3.4 @ 12m depth (28 d)</td>
<td></td>
</tr>
</tbody>
</table>

3. Required Properties For DSM

For DSM, the binders would seek to improve the following treated soil properties:

- Strength – cohesion (c) and internal angle of friction (φ) which determines both shear strength (s) and compressive strength (q);
- Compressibility – Youngs Modulus (E) and Constrained Modulus (M) which determines settlement behaviour;
- Plasticity – Atterberg limits which determines the critical stages of fine-grained soil state and behaviour;
- Dynamic – Shear Modulus (G) and Damping Ratio (D) which affects resistance to liquefaction and soil structure interaction (SSI) effects when dynamic loading conditions occur.

The effectiveness of the binder reaction with soil is significantly influenced by:

- Binder type and insertion methods – WDSM in jetted slurry form / DDSM by compressed air;
- Mixing methods – binder mix consistency with the soil;
- Binder dosage applied (higher dosage up to an optimum level lead to greater strength);
- Type and content of reactive pozzolanic material in soil – i.e. silica / aluminate material in the soil. High organic matter content in soils like peat can retard cementitious reaction process. Pozzolanic reactions will not initiate or be effective if there are insufficient reactive silica or aluminates;
- Soil temperature & pH – higher temperature increases rate of reaction. High acidity inhibits reaction rate;
- Allowable curing period – strength increases with curing time;
- Specific surface of binder particle – A higher specific surface causes faster reaction to take place (where smaller particles have a higher specific surface).

![Diagram of DSM Binders methodology]

Figure 1. DSM Binders methodology

![Graph showing normalized strength (qt/q28) development with curing time]

Figure 2. Normalized Strength (qt/q28) development with curing time

4. Traditional Binder Materials

4.1. Reaction Mechanism

Traditional cementitious binder materials, OPC, lime or a combined mixture of both materials are still the most commonly used in DSM. Lime, in the form of calcium oxide (CaO), is derived from crushing and heating limestone...
(CaCO₃) at over 1000 °C. When added to soil, CaO undergoes a calcareous reaction with the water content in soil to form slaked lime or hydrated lime, otherwise known as calcium hydroxide (Ca(OH)₂) [9]. Combining with water in soil results in hydration which reduces the water content in the soil which leads to better soil stability. Strength development is derived from secondary reactions between any pozzolanic materials mixed with lime or as particles in the soil. These secondary pozzolanic reactions form C-S-H, C-A-H and C-A-S-H which are the main contributing components to strength gain. Other benefits of lime binders come from the heat release from exothermic CaO / H₂O reaction of CaO and increase in pH due to Ca (OH)₂ formed which improves the pozzolanic reaction rate.

Cement (OPC) reacts hydraulically with water to form a paste which binds the soil and other binder particles together into a hardened mass. Strength development comes from formation of C-S-H and C-A-H due to reaction of C₃S, C₃S and C₃A components of cement with water.

4.2. Lime Binders

Lime as a binder in stabilization has been used in deep soil mixed columns for loose clays, silts and peat soils since the 1970s in Scandinavia. Undrained shear strengths for stabilized clay under favorable conditions of 10 to 50 times original soil strength have been recorded after one year [10, 11]. Studies were also conducted on the engineering characteristics of lime stabilized organic soils [12] and design principles for stabilization using lime columns have been established [10]. Lime neutralizes the acidity of organic matter in soils and improves soil plasticity and shear strength. However, strength development in lime columns is still susceptible to several factors:

- Soil temperature – lower temperatures slows the reaction considerably;
- Low pH in soil retards reaction and leads to long-term deterioration of the soil-lime column;
- Sulphate content – high soil sulphate content interferes with normal pozzolanic reactions between lime and any pozzolanic particles in the soil. This is similar to cementitious hydraulic reactions when low-density ettringite forms in the presence of sulphate ions leading to expansion and cracking in the brittle soil-lime soil mix [13].

<table>
<thead>
<tr>
<th>Binder Type</th>
<th>Binder Component</th>
<th>Binder comp. % (wt. binder)</th>
<th>Reaction Equation</th>
<th>Reaction time</th>
<th>Effect</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>C₃S</td>
<td>55</td>
<td>2C₃S + 7H → C₃S₆H₄ + 3CH</td>
<td>rapid</td>
<td>Early strength gain</td>
</tr>
<tr>
<td></td>
<td>C₂S</td>
<td>18</td>
<td>2C₂S + 5H → C₂S₄H₄ + CH</td>
<td>slow</td>
<td>Long term strength gain</td>
</tr>
<tr>
<td></td>
<td>C₃A</td>
<td>10</td>
<td>2C₃A + 21H → C₃AH₁₃ + C₃AH₈ → 2C₃AH₆ + 9H</td>
<td>rapid</td>
<td>Early set (hardening)</td>
</tr>
<tr>
<td></td>
<td>C₃A + (C₅S₂H₂)</td>
<td>10</td>
<td>2C₃A + 3C₅S₂H₂ + 26H → C₃A₆S₃H₁₂</td>
<td>rapid</td>
<td>Slows down set reaction of C₅S₂, C₃S</td>
</tr>
<tr>
<td></td>
<td>C₆AF + (C₃S₂H₂)</td>
<td>8</td>
<td>3C₆AF + 12C₃S₂H₂ + 110H → 4[C₃A₆S₃H₁₂] + 2 AF₃, t</td>
<td>slow</td>
<td>Slows down set reaction of C₃S, C₅S</td>
</tr>
</tbody>
</table>

Lime

<table>
<thead>
<tr>
<th>Binder Type</th>
<th>Binder Component</th>
<th>Binder comp. % (wt. binder)</th>
<th>Reaction Equation</th>
<th>Reaction time</th>
<th>Effect</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CaO</td>
<td>100</td>
<td>CaO + H₂O → Ca (OH)₂</td>
<td>rapid</td>
<td>increases pH to ~ 12.5; reduce water content in soil</td>
</tr>
<tr>
<td></td>
<td>N/A</td>
<td>N/A</td>
<td>Ca (OH)₂ + pozzolans in soil + H₂O → C-S-H and/or C-A-S-H</td>
<td>slow</td>
<td>Long term strength gain</td>
</tr>
</tbody>
</table>

1 Adapted from Ahlberg and Johansson (2005)
2 CaO = Calcium Oxide ; Ca(OH)₂ = Calcium Hydroxide ; CH = Calcium Hydrate ; C-S-H = Calcium Aluminate Silicate Hydrate (ettringite) ; C-A-H = Calcium Silicate Hydrate (C-SH) ; C-A-H = Calcium Silicate Aluminate ; AF₃ = Alumino Ferrite mono and tri phases; C₃S = Tricalcium Silicate ; C₅A = Tricalcium Aluminate ; C₆AF = Tetra calcium Alumino Ferrite ; Gypsum = C₃S₂H₂

4.3. Cement Binders

Cement has been the most researched and widely utilized binder material for deep soil mixing. Deep cement columns in Japan have typically achieved over 1 MPa UCS by wet method installation and typically, 500 kPa by dry method installation methods [14]. Cement offers greater soil mechanical improvement (shear strength and compressibility) over lime [9, 15]. The design methodology for combined lime/cement and cement columns have been developed for soft clays [16, 17].

For peat soil, the application of deep soil mix columns was investigated with a series of simple scale models in the field [18–20]. Huat et al. [21] conducted a series of tests on various proportions of cement and lime binders on peat soil, showing increasing strength improvement in the order of 2 to 3 times as binder % increases. The implementation of deep mixed methods cement columns in peat soils was also studied in the lab by a series of Rowe cell tests [22]. In soft organic soils (e.g. peat), the organic matter in the soil inhibited cementitious reaction [23]. Research by Hebib and Farrell [24] supported the finding that the combination of high organic content, lack of solid and pozzolanic particles, acidic media (from humic acid) and high water: solids ratio all seek to impede efficient hydration of cement in peat soils.
Investigation of the influence of cement binder dosage [25, 26] showed an increase in cement ratio leads to improved compressibility characteristics of soft clay and peat soil.

Longer curing periods also lead to greater strength development in cement-treated soil. Logarithmic models were widely used to predict strength gain over time [4, 27]. A shortcoming of a logarithmic model was that it implied indefinite strength increase with time. Logically, cement hydration should cease when reaction products are consumed. This led to an improved hyperbolic function model on strength development over time [28, 29].

5. Pozzolanic Binder Materials

5.1. Reaction Mechanism

Pozzolans, are defined as "siliceous or siliceous and aluminous materials which in themselves possess little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties" [30]. The chemical reaction between these siliceous and/or siliceous-alumina rich components, calcium hydroxide and water, is called the pozzolanic reaction. Also known as supplementary cementitious materials (SCM), pozzolans can act as a beneficial additive to cement with various performance enhancing effects for soil stabilization. Pozzolans can be categorized as:

- Natural Pozzolans – either of volcanic (unaltered pyroclastic materials e.g. vitreous pumices / ashes or altered pyroclastic materials like zeolitized tuffs etc.) or sedimentary origin (e.g. chemical sediments e.g. diatomaceous earth or detrital sediments e.g. clays, shales etc.);
- Artificial Pozzolans – e.g. Blast Furnace slag (GGBS), Fly Ash (FA), Silica Fume (SF) and burned organic matter residue with significant siliceous/aluminous materials – e.g. Rice Husk Ash (RHA).

Cementitious hydration, as mentioned earlier, not only produces Calcium Silica Hydrate (C-S-H) but also, as a byproduct, Ca (OH)_2 (up to 25% of hydrated Portland cement). The dissolved SiO_2 (combined with water to form silicic acid, H_4SiO_4) and Al_2O_3 in pozzolans react with dissolved Ca^{2+} and (OH)^- ions from the Ca(OH)_2 to produce both C-S-H and Calcium Aluminate Hydrate C-A-H [31].

\[
\begin{align*}
\text{Ca}^{2+} + 2\text{(OH)}^+ + \text{SiO}_2\text{(in pozzolans)} & \rightarrow \text{C-S-H} \\
\text{Ca}^{2+} + 2\text{(OH)}^+ + \text{Al}_2\text{O}_3\text{(in pozzolans)} & \rightarrow \text{C-A-H}
\end{align*}
\]

A similar mechanism that may reside for pozzolans in soils occurs when lime / cement is added. The derived Ca(OH)_2 is transported via water within the soil to combine with aluminate and/or silicate clay minerals in the soil [32]. However, enough free calcium ions and a pH level above 12 is needed (to maintain solubility of silicon and aluminium ions) to initiate and maintain the pozzolanic reaction [33]. This can be provided from Ca (OH)_2 that is derived from addition of either lime or cement.

Field test results from DSM samples [4, 34] and laboratory results from cement-treated soil [27] show significant long-term strength gain (up to 2.1 strength increase over 28 day strengths) well beyond the short term cement hydration phase [35]. This long term strength gain has been attributed to secondary pozzolanic reactions taking place in the soil as a result of the right conditions from cement hydration – e.g. high pH, presence of Ca^{2+} ions / (OH)^- ions and pozzolanic materials in the soil [36, 37].

Pozzolanic reactions occur over longer timescales (months to years) [9, 31]. Correia et al. [38] proposed a simplified method to predict the UCS at 28 days for cement and pozzolanic stabilized clays based on soil liquidity index, specific binder type and content. The effectiveness of pozzolans (reaction and strength) is related to:

- Reactivity with lime (CaO) – The reactivity of mineral admixtures reactivity of the pozzolanic materials can be determined by the Chapelle (which determines the rate of lime (CaO) consumption) or similar tests [39];
- The proportions of the pozzolan phase state (crystalline / vitreous or amorphous) which affects strength [9, 40] – higher amorphousness leads to greater reactivity. Reactivity-based on lime consumption per mass of the amorphous
phase of pozzolans using Reitveld refinement method of XRD analysis interpretation allows consideration of vitreous or amorphous phase in the pozzolanic material and more accurate determination of the effectiveness of a pozzolan [41].

- The particle size and associated specific surface of the pozzolan which affects reactivity [40] – the higher the specific surface, the greater the reactivity.

### Table 3. Mineral composition of various pozzolanic materials

<table>
<thead>
<tr>
<th>Binder</th>
<th>Mineralogy (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>CaO</td>
</tr>
<tr>
<td>OPC ³</td>
<td>67.7</td>
</tr>
<tr>
<td>Lime (CaO)</td>
<td>100.0</td>
</tr>
<tr>
<td>FA - Class C ²</td>
<td>14.8</td>
</tr>
<tr>
<td>FA - Class F ²</td>
<td>2.4</td>
</tr>
<tr>
<td>GGBS ²</td>
<td>42.0</td>
</tr>
<tr>
<td>SF</td>
<td>1.0</td>
</tr>
<tr>
<td>RHA³</td>
<td>0.0</td>
</tr>
<tr>
<td>K⁴</td>
<td>0.3</td>
</tr>
<tr>
<td>MK⁴</td>
<td>0.0</td>
</tr>
</tbody>
</table>

Others include: SO₃, Na₂O, K₂O, TiO₂, MnO₂, etc.
Values obtained from: ³ [42]; ² [43]; ⁴ [44] (Table 1).

### Table 4. Typical Particle size / Specific Surface of various pozzolanic materials

<table>
<thead>
<tr>
<th>Physical Property</th>
<th>Binder Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>OPC ²</td>
</tr>
<tr>
<td>Particle size range (μm)</td>
<td>10 – 40</td>
</tr>
<tr>
<td>Specific Surface (m²/g)</td>
<td>1.75</td>
</tr>
</tbody>
</table>


### 5.2. Fly Ash (FA)

Fly Ash (also known as “pulverized fuel ash”, PFA or FA) is residue composed of pulverized coal, discharged from the combustion chamber by exhaust gases in coal-fired power plants. There are two classes of fly ash:

#### Table 7. Chemical content of different Fly ash classes [58]

<table>
<thead>
<tr>
<th>FA Class</th>
<th>CaO</th>
<th>SiO₂</th>
<th>Al₂O₃</th>
<th>Fe₂O₃</th>
<th>LOI</th>
<th>sulphates</th>
</tr>
</thead>
<tbody>
<tr>
<td>F (anthracite</td>
<td>bituminous)</td>
<td>1 – 12</td>
<td>20 – 60</td>
<td>5 – 35</td>
<td>10 – 40</td>
<td>0 – 15</td>
</tr>
<tr>
<td>C (sub-bituminous)</td>
<td>5 – 30</td>
<td>40 – 60</td>
<td>20 – 30</td>
<td>4 – 10</td>
<td>0 – 3</td>
<td>0 – 2</td>
</tr>
<tr>
<td>C (lignite)</td>
<td>15 – 40</td>
<td>15 – 45</td>
<td>20 – 25</td>
<td>4 – 15</td>
<td>0 – 5</td>
<td>0 – 10</td>
</tr>
</tbody>
</table>

Class F FA requires an activator or cementing agent (as it contains less lime (CaO) content) mixed with water. It can form a geopolymer by combining with sodium silicate/ sodium hydroxide [59]. When utilized to stabilize soft organic soils, the addition of fly ash increases soil resilient modulus (Mr) from zero Mr without a binder to 10-100 MPa depending on % of binder used and improves unconfined compressive strength (UCS) from original untreated at 15 kPa to > 100 kPa with fly ash [60]. Similarly, for tropical peat soils, the mixing of Pond Ash (PA) binder also increases UCS. 20% PA dosage lead to doubling UCS of the original peat soil [61,62].

Significant properties enhancements were reported for fly ash binders of organic (36.9% organic content) soil from Khulna, Bangladesh, notably in the liquid (LL) and plastic limits (PL) as well as dry density leading to increased UCS [63]. Two types of Fly Ash – Class C and Class F (ASTM C 618-2017a), were tested whereby, Class C fly ash achieved noticeable higher strength gains over Class F type. Stabilized soil pH values also increased because of binder mixing.
Table 5. Summary of some research on pozzolanic binders – Fly Ash (FA) / Ground Granulated Blast Slag (GGBS) / Silica Fume (SF)

<table>
<thead>
<tr>
<th>Research</th>
<th>Soil Type / soil properties</th>
<th>Pozzolan binder</th>
<th>Cementitious binder</th>
<th>Findings</th>
<th>Proportion / improved Properties</th>
<th>UCS¹ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[46]</td>
<td>• Peat</td>
<td>• Fly Ash (F1);</td>
<td>• OPC;</td>
<td>• Best results obtained with GGBS and bypass ash (F1)</td>
<td>• 200 kg/m³ OPC / FA OPC: F1=50:50</td>
<td>382 (26 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• GGBS;</td>
<td>• Lime;</td>
<td>• Less clay particles in peat → no significant pozzolanic reaction</td>
<td>• 200 kg/m³ OPC / GGBS OPC: GGBS=50:50</td>
<td>1340 (30 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Sand (FS);</td>
<td>@ (50:50 / 60:40 for cement: pozolan) % wt. binder</td>
<td>• OPC more effective than lime in peat soils due to humic acids</td>
<td>• 200 kg/m³ OPC / 100 kg/m³ FS</td>
<td>792 (30 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>@ 100–250 kg/m³</td>
<td></td>
<td>• Addition of sand filler acts as stiffener</td>
<td>• 300 kg/m³ OPC</td>
<td>1250 (30 d)</td>
</tr>
<tr>
<td>[47]</td>
<td>• Gyttja</td>
<td>• Fly Ash (FA-F)</td>
<td>• OPC;</td>
<td>• High silica % contributes to pozzolanic reaction</td>
<td>• 30% FA / OWC=47% CaO: SiO₂=21.3:40.2</td>
<td>148 (7 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(55% SiO₂ / 9% CaO);</td>
<td>• OPC;</td>
<td>• Clay soils (OL + ML) &lt;30 kPa untreated to &gt; 400 kPa (30% FA)</td>
<td>• 30% FA / OWC=28% CaO: SiO₂=23.3:31.1</td>
<td>411 (7 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>• Fly Ash FA-C</td>
<td>• OPC;</td>
<td>• Peat soil (Pt) from &lt;15 kPa untreated to &gt; 100 kPa (30% FA). Higher SiO₂ % leads to higher UCS</td>
<td>• 30% FA / OWC=21% CaO: SiO₂=21.3:40.2</td>
<td>490 (7 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(40% SiO₂ / 24% CaO);</td>
<td>• OPC;</td>
<td>• Optimum water content (OWC) applies for different soils / binders</td>
<td>• OPC; F1=50:50</td>
<td>1861 (28 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>@ 10–30 % wt. soil</td>
<td>• OPC;</td>
<td>• Optimum CaO / SiO₂ ratio established at 0.5 - 0.8</td>
<td>• OPC; GGBS=50:50</td>
<td>1340 (30 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• OPC;</td>
<td>• MDD, UCS increase as binder % increase</td>
<td>• OPC; 200 kg/m³ OPC</td>
<td>792 (30 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• OPC;</td>
<td>• FSI, LL, PI decrease as binder % increase</td>
<td>• OPC; 300 kg/m³ OPC</td>
<td>1250 (30 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>• OPC;</td>
<td>• For UCS, optimum at 10% GGBS</td>
<td>• OPC; 400 kg/m³ OPC</td>
<td>N/A</td>
</tr>
<tr>
<td>[48]</td>
<td>• Clay (PH)</td>
<td>• GGBS;</td>
<td>• OPC;</td>
<td>• UCS and CBR measured</td>
<td>• 15% OPC P=false</td>
<td>1861 (28 d)</td>
</tr>
<tr>
<td></td>
<td>Pb=54%</td>
<td>@ 0–15 % wt. soil</td>
<td>• OPC;</td>
<td>• For 5-15% cement, 10% dose SF → higher UCS</td>
<td>• 10% GGBS P=false</td>
<td>779 (28 d)</td>
</tr>
<tr>
<td></td>
<td>FSI=154%</td>
<td></td>
<td></td>
<td>• For 20-50% cement, 5% SF → higher UCS</td>
<td>• OPC; 5% SF CBR=21.4%</td>
<td>320 (90 d)</td>
</tr>
<tr>
<td></td>
<td>q=169 kPa</td>
<td>• OPC;</td>
<td>• OPC;</td>
<td>• CBR increases as binder content increases</td>
<td>• OPC; 5% SF CBR=21.4%</td>
<td>620 (90 d)</td>
</tr>
<tr>
<td>[49]</td>
<td>• Peat</td>
<td>• Silica Fume (SF)</td>
<td>• OPC;</td>
<td>• UCS and CBR measured</td>
<td>• 5% OPC / 10% SF CBR=21.4%</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>CBR=0.78%</td>
<td>@ 5–10 % wt. OPC</td>
<td>• OPC;</td>
<td>• For 5-15% cement, 10% dose SF → higher UCS</td>
<td>• 50% OPC / 5% SF CBR=21.4%</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td></td>
<td>@ 5–50 % wt. soil</td>
<td>• OPC;</td>
<td>• For 20-50% cement, 5% SF → higher UCS</td>
<td>• OPC; 5% SF CBR=21.4%</td>
<td>N/A</td>
</tr>
<tr>
<td>[50]</td>
<td>• Peat</td>
<td>• Silica Fume (SF)</td>
<td>• OPC;</td>
<td>• c and φ increases as binder % increases</td>
<td>• 15% OPC c=250 kPa / φ = 22°</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>q=29.5 kPa</td>
<td>@ 5–10 % wt. OPC</td>
<td>• OPC;</td>
<td>• Settlement reduced by 35% with 5% cement</td>
<td>• 15% OPC c=280 kPa / φ = 26°</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>c=0.01 kPa</td>
<td>@ 5–50 % wt. soil</td>
<td>• OPC;</td>
<td>• Presence of any type of small particle (known as particle packing or micro filling) will improve the strength in the presence of cement</td>
<td>• OPC</td>
<td>N/A</td>
</tr>
</tbody>
</table>

¹ extracted from Figures in articles; ² prepared from Bentonite and sand mixture
### Table 6. Summary of some research on pozzolanic binders – Rice Husk Ash (RHA) / Kaolin (K) / Metakaolin (MK)

<table>
<thead>
<tr>
<th>Research</th>
<th>Soil Type / soil properties</th>
<th>Pozzolan binder</th>
<th>Cementitious binder</th>
<th>Findings</th>
<th>Proportion / improved Properties (% / kg/m³ soil)</th>
<th>UCS¹ (kPA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>[51]</td>
<td>• Residual</td>
<td>RHA 0–25% OPC</td>
<td>OPC; 0–14% soil</td>
<td>• UCS and CBR increase as binder increases</td>
<td>4% OPC / 5% RHA</td>
<td>250 (28 d)</td>
</tr>
<tr>
<td></td>
<td>q=100 kPa CBR=3.8%</td>
<td>@ 0–25% wt. OPC</td>
<td>@ 0–14 % wt. soil</td>
<td>• RHA addition requires less cement to achieve same UCS compared to only cement-stabilized soils. RHA cannot be used by itself to increase UCS</td>
<td>Optimum for CBR CBR=60%</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>• RHA</td>
<td>• OPC; 0–14% soil</td>
<td>• RHA increases resistance to reduced UCS when soaked - optimum binder content at 10% RHA</td>
<td>• RHA addition requires less cement to achieve same UCS compared to only cement-stabilized soils. RHA cannot be used by itself to increase UCS</td>
<td>8% OPC / 28% RHA</td>
<td>1200 (7 d)</td>
</tr>
<tr>
<td></td>
<td>@ 0–25% wt. OPC</td>
<td></td>
<td></td>
<td>Optimum for CBR CBR=60%</td>
<td></td>
<td>N/A</td>
</tr>
<tr>
<td>[52]</td>
<td>• Clay (CI)</td>
<td>RHA 0–20% soil</td>
<td>–</td>
<td>• UCS increases as % lime increases</td>
<td>5% RHA CBR=4.8% (soaked)</td>
<td>1212 (28 d)</td>
</tr>
<tr>
<td></td>
<td>q=130 kPa CBR=2.6% (soaked)</td>
<td>@ 0–20% wt. soil</td>
<td></td>
<td>• For UCS, optimum at 5% RHA</td>
<td></td>
<td></td>
</tr>
<tr>
<td>[53]</td>
<td>• Silty Clay (ML)</td>
<td>RHA 3–7% Lime</td>
<td>Lime Ca(OH); 4, 6% soil</td>
<td>• UCS Increases as % lime increases</td>
<td>4% lime / 5% RHA</td>
<td>N/A</td>
</tr>
<tr>
<td></td>
<td>c=35 kPa φ = 35° CBR=8.9%</td>
<td>@ 3–7% wt. Lime</td>
<td>@ 4, 6 % wt. soil</td>
<td>• UCS peaks (&gt;50%) at 15–20% RHA with 5% lime thereafter decreases with as RHA % increases</td>
<td>Optimum for CBR CBR=54%</td>
<td>N/A</td>
</tr>
<tr>
<td>[54]</td>
<td>• Silty Sand (SW-SM)</td>
<td>RHA 5–20% Lime</td>
<td>Lime CaO; 3, 5 % wt. Soil</td>
<td>• UCS Increases as % lime increases</td>
<td>5% lime / 5% RHA</td>
<td>250 (28 d)</td>
</tr>
<tr>
<td></td>
<td>q=8.2–15 kPa</td>
<td>@ 5–20% wt. Lime</td>
<td>@ 3, 5 % wt. Soil</td>
<td>• UCS peaks (&gt;50%) at 15–20% RHA with 5% lime thereafter decreases with as RHA % increases</td>
<td></td>
<td></td>
</tr>
<tr>
<td>[55]</td>
<td>• Peat</td>
<td>RHA 5–10% Lime</td>
<td>Total binder @ 300 kg/m³ + Silica Sand @ 596 kg/m³ OPC @ 90% wt. total Binder</td>
<td>• UGS increases as binder dosage increases</td>
<td>90% OPC / 10% K + add. 4% CaCl₂ @ 300 kg/m³ total binder</td>
<td>485 (7 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>@ 5, 10% wt. overall Binder</td>
<td></td>
<td>• Increasing silica sand as filler leads to UCS increase</td>
<td></td>
<td></td>
</tr>
<tr>
<td>[56]</td>
<td>• Clay</td>
<td>Metakaolin (MK) 3–10% soil</td>
<td>OPC 27–50% soil</td>
<td>• UGS increases as MK / OPC dosage increases and is high enough to be considered as (soilcrete) structural material</td>
<td>27% OPC / 3% MK</td>
<td>17 MPa (28 d)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>@ 3–10% wt. soil</td>
<td></td>
<td>• Youngs modulus (E) improves also</td>
<td>40% OPC / 10% MK</td>
<td>57 MPa (28 d)</td>
</tr>
<tr>
<td>[57]</td>
<td>• Clay (CH)²</td>
<td>Metakaolin (MK) 3–10% soil</td>
<td>–</td>
<td>• Clay (CH)²</td>
<td>10% MK</td>
<td>630 (1 d)</td>
</tr>
<tr>
<td></td>
<td>c=94.76 kPa q=190 kPa</td>
<td>@ 3–10% wt. soil</td>
<td></td>
<td></td>
<td>c=315 kPa</td>
<td>@ 2.8% strain</td>
</tr>
</tbody>
</table>

¹ extracted from Figures in articles; ² prepared from Bentonite and sand mixture.
5.3. Ground Granulated Blast Slag (GGBS) or Slag

Slag, alternately known as Ground Granulated Blast Furnace Slag (GGBS or GGBFS) is formed as a by-product from iron production by quenching molten iron slag from a blast furnace to produce a glassy, granular material that is dried and ground into a fine powder. Chemical composition varies with the composition of raw materials in iron production and is mainly composed of – CaO [typically 30-50%]; SiO₂ [from 28-38%]; Al₂O₃ [8-24%] and MgO [between 1-18%]. Properties of GGBS when blended with cement and in soil stabilization [64] that are useful in DSM are:

- Lower early temperature rise – which reduces the risk of thermal cracking for mass pours;
- Reduced soil plasticity (lower PI) and quc increase in clays;
- Reduced risk of alkali-silica reaction (ASR) – a swelling reaction over time in concrete between highly alkaline cement paste and reactive non-crystalline (amorphous) silica in aggregates;
- Greater resistance to sulphate attacks and to chloride infiltration in cement-GGBS mix;

GGBS is considered a latent hydraulic material and requires activation (by hydraulic binder material like cement or lime or alternatively by using alcalis and sulphates) before it can react with water in the soil [9]. The reactivity of the GGBS also depends on its phase state which results from the rate of cooling after leaving the furnace. Rapid cooling leads to an amorphous, highly reactive slag while slow cooling would lead to crystalline inert slag, which is unsuitable as an additive for stabilization in deep soil mixing [9].

Typically, GGBS would be utilized as a partial substitute to a primary cementitious binder (e.g. OPC) with substitution ratios that have been researched from 75% OPC:25% GGBS to 50% OPC:50% GGBS for total dosages between 75 to 300 kg/m² [65, 66].

Studies on clay & silt in Sweden showed slower strength development with combinations of slag-cement-slag ratio binder mixes compared to cement alone [11]. This is due to the slower reaction of GGBS, which has a lower CaO/SiO₂ ratio to cement. Although better reaction product quantities from GGBS-lime mix vs GGBS-cement mix were reported, they were still inferior to lime-cement mix or lime binder in clay [11]. Comparison between cement and GGBS as binders to expansive clays showed that 10% GGBS (optimum) (779 kPa) achieved an equivalent 28 day UCS to 5% cement content (764 kPa) [48].

For peat soil, research results showed lower compressive strength at 7 days curing for 75% OPC:25% GGBS binder mix compared to 100% OPC binder mix with 25% silica sand [67]. Axelsson et al. [68] concluded that cement: GGBS binder mix provided better results for compressive strength than for cement binder alone in peat.

5.4. Silica Fume (SF)

Silica Fume (SF) is a pozzolanic by-product material derived from the production of silicon metal/ferro silicon alloys in smelters using electric arc furnaces – e.g. aluminium and steel production, computer chip fabrication plants, silicone production etc. SF typically consists of spherical particles with an average particle diameter of 150 nm and is typically 85-97 % SiO₂ with less than 1% CaO.

Because of its chemical composition and fineness, SF only enhances the properties of concrete and is not meant as a replacement material for Portland cement such as Fly Ash or GGBS. The major improvement effect on fresh concrete is a more cohesive slurry mix leading to:

- Improved bond strength; little or no bleeding in the concrete;
- Reduced permeability which improves durability and resistance to chloride and sulphate attack;
- Enhanced UCS and E for the treated soil.

The small particle size of SF also leads to greater surface area and therefore allows the SiO₂ to react more readily with Ca(OH)₂ in the pozzolanic reaction to produce C-S-H, leading to improved strength properties.

Because it is not utilized as a replacement for cement, SF can only be considered as a minor additive to cement in soil stabilization. Typical SF content may range between 10 to 75 kg/m² or 4 - 15% of cement content by weight.

Research performed on cement with SF (5-10% by weight cement) on peat soil showed that the addition of SF has contributed to settlement reduction and increased bearing capacity [69, 70]. Optimum dosage up to 5% cement content in soil was achieved with 10% SF, thereafter, 5% SF by cement weight was more effective.
5.5. Rice Husk Ash (RHA)

Rice Husk Ash (RHA) is formed by burning, at a specific temperature range, rice hulls / husk which are separated from rice grains as a by-product in rice milling. About 10^8 tons of rice husk are generated annually in the world [71]. The ash residue from the combustion is a potential source of non-crystalline / amorphous reactive silica (of up to 95%) [72]. Typical composition of RHA (depending on specific combustion conditions) can be 88 [39] – 99 % SiO_2 [73]. RHA has the highest specific surface area amongst the pozzolanic materials covered in this review [39].

Again, a small particle size leads to greater surface area and therefore allows improved reaction by SiO_2 with Ca (OH)_2 to produce strength enhancing C-S-H.

The type of RHA used is important – the amorphous form of SiO_2 has higher pozzolanic reactivity compared to the crystalline form of SiO_2 [74]. The composition of SiO_2 is dependent on the combustion process which leads to the structural transformation of the SiO_2 in the ash residue. High-temperature combustion produces more crystalline forms of SiO_2 as opposed to the low temperature combustion process. This can be differentiated through colour – high temperature ash (>900 °C) produces fully crystalline silica and is white / pink in colour whereas lower temperature ash (500-700 °C) is darker (grey to black) with more amorphous silica that is more reactive [75]. This is counteracted by less SiO_2 derived from lower furnace temperature furnaces vs higher furnace temperature. RHA with lower carbon content produces higher pozzolanic activity [76].

Basha et al. [77] tested the addition of RHA with cement binder on residual granite soil. Like SF, RHA cannot be utilized as a full replacement for cement but as a supplement to the cement to achieve multiplier effects on enhancing soil properties. This can be seen when RHA only treated soil encountered decreased CBR values with increasing % RHA vs increasing CBR with an increase in % cement.

RHA does not appear to improve all types of soils. Alhassan [71] utilized RHA as binder material sans cement for tests on ferruginous tropical soils and recorded insignificant bearing capacity improvement over original soil.

Research on clayey silt showed a 63% increase in compressive strength after 7 days curing over original soil with an optimum 15% by soil weight [78]. Choobasti et al. [79] investigated the influence of RHA as an additive binder to slaked lime for silty clay soils and proposed an optimum mix of 4% lime and 5% RHA to maximize e & φ properties in the soil. Roy [80] recommended optimizing at 6% cement and 10% RHA for clay soils. Research by Rahman et al. [81] on the addition of RHA binder to silty sand, showed shear strength improving as RHA % increases.

5.6. Kaolin (K)

Kaolin (K), also known as china clay is a naturally occurring soft white clay of which the mineral kaolinite is obtained from. The chemical formula for Kaolinite is Al_2Si_2O_5(OH). Kaolinite is one of the most common minerals; it is mined in various countries throughout the world [82].

Chemical composition of Kaolin powder consists of 57.6% SiO_2 | 37.8% Al_2O_3 | 0.9% Fe_2O_3 | 0.6% MgO being the main constituents [42]. Being both a siliceous and aluminous material, Kaolin is a partial pozzolanic additive for cement reacting to form C-S-H and C-A-H products [83].

Wong et al. [84] proposed kaolin mixed with lime / cement for use as partial replacement and pozzolanic additive in stabilized peat columns. Additionally, kaolin and silica sand (as a filler) on peat soils demonstrated combined action of hydrolysis of the cement, the pozzolanic reaction of kaolin, and the filler effect of well-graded silica sand in the soil.

5.7. Metakaolin (MK)

Metakaolin (MK), is produced by heating kaolin clay to high temperatures (calcination in the range starting from 700-850 °C) [85]. The chemical formula for Metakaolinite mineral (MK) is Al_2O_3, 2SiO_2[56]. Metakaolin (MK) powder chemical composition is similar to Kaolin at 53.2% SiO_2 | 43.9% Al_2O_3 | 0.38% Fe_2O_3 | 0.02% CaO being the main constituents [86] depending on the supplier. Metakaolin has a specific surface of 12680 cm^2/g [86] which is almost 4 times greater than cement, and hence has greater reactivity.

Considered to have twice the reactivity of most other pozzolans, metakaolin is a valuable admixture for concrete/cement applications. When replacing Portland cement with 8–20% (by weight) metakaolin produces a concrete mix which exhibits favorable engineering properties that includes: the filler effect, the acceleration of OPC hydration, and the pozzolanic reaction. The filler effect is immediate, while the effect of pozzolanic reaction occurs between 3 and 14 days. Kolovos et al. [56] investigated soil-crete mixtures modified with metakaolin. MK addition to binder mix improved UCS significantly over just cement binder in clay through a reduction in porosity and microcracking and formation of dense cement gel.
6. Filler Materials

6.1. Fine Sand

Fine sand acts as an inert filler and does not contribute to cement hydration or pozzolanic reactions. However, it provides a structure for the binder particles to attach to and form a load bearing stabilized soil and also contributes to densification by filling the void spaces in the soil during stabilization [55]. Tests with increasing sand content in highly compressible soil – e.g. peat, has shown improved settlement characteristics [87]. Wong et al. [88] researched the effect of Kaolin (as Supplementary Cementitious Material) and Silica sand (as a filler) on peat soils, Klang, Selangor whereby:

- An increase in silica sand (0 – 596 kg/m$^3$) led to increased UCS (from 175 kPa to 460 kPa);
- Threshold silica sand dosage of 460 kg/m$^3$ is suggested to achieve minimum UCS = 345 kPa;
- Increases density and reduces the porosity of stabilized soil;

However, the fine sand does not contribute to secondary pozzolanic reaction (with nano materials and cement) or geo-polymerization (with the right chemical additive), being non-crystalline (amorphous) silica.

7. Discussion

Adding pozzolan materials to lime / cement binder mix can be effective when treating soils with little or no pozzolanic particles since follow on strength development (which may account for another 50-100% of 28-day strength) beyond short term hydration is due to pozzolanic reactions [37]. However, pozzolans are ineffective by themselves and require Ca (OH)$_2$ presence in aqueous solution to supply enough quantities of Ca$^{2+}$ and OH$^-$ for the pozzolanic reaction to take place. The Ca (OH)$_2$ is usually provided by a cementitious binder.

Pozzolans may be divided into two categories:

- Siliceous materials (> 90% SiO2) – e.g. RHA, SF, FA etc.;
- Aluminosilicate materials (with both SiO2 and Al2O3) – e.g. GGBS, K, MK.

From the compilation of research results for different pozzolan binders in Tables 5 and 6, there is a clear trend showing aluminosilicate pozzolans producing greater compressive strength in combination with cementitious binders. The lower proportion of amorphous form of siliceous materials, which determines pozzolanic reactivity, also contributes to the disadvantage to aluminosilicates.

Alumino silicate pozzolans exhibit higher initial rates of reaction over siliceous pozzolans due to Al$_2$O$_3$, which form C-A-S-H and C-A-H [86]. The greater specific surface area and smaller particle size of MK would lead to greater reaction rate over GGBS.

With pozzolans, there is an optimum dosage limit that applies, whereupon performance peaks. For example, the 10% optimum dosage by weight with MK has been observed in several researches [56, 57, 86]. This suggests a required concentration of Ca (OH)$_2$ greater than what is provided from the cement hydration, that must be maintained in relation to the pozzolan in order to retain rate of pozzolanic reaction. The effect from more Ca (OH)$_2$ in order to increase effectiveness of a higher pozzolan dosage limit by adding lime should be investigated further.

In terms of compressive strength improvement, the optimum dosage for some siliceous binder types (15 to 28% for RHA [54,77]) differ from aluminosilicate binders (5 to 10% replacement dosage for GGBS [48] and MK [56,57,86]). A comprehensive comparison of optimum dosage between the two groups of pozzolans when combined with cementitious binder for different soil types e.g. clay, silts, sands, organic soils etc. would be useful.

Frias and Cabrera [35] noted also that a higher water/binder (w/b) ratio may accelerate the completion of pozzolanic reaction for MK. Thus, the long term slow pozzolanic reaction beyond typical 28-day cement binder hydration may be accelerated with a combination of w/b ratio and use of MK blended-cement upon further investigation. It is noted that w/b ratios can only be controlled with the WDSM method.

The actual beneficial effect of filler materials has been sparsely researched. A good starting point would be applying research performed on effect of aggregate in concrete as a starting point to how filler materials can provide solid particles for binder paste to adhere to. Filler materials mixed with binders would be feasible if injected as a slurry in WDSM method for organic and very soft soils. They can act as economic inert substitutes when lower strengths are only required.

Finally, the influence of binders on dynamic properties of treated soil in context of DSM is not well covered in literature. As compressive strength, modulus of elasticity and shear modulus of soil are co-related, comprehensive investigation should be carried out to determine the extent of beneficial influence by binder treated soil when laid in DSM columns.
8. Conclusions

This review paper explains the reaction mechanism and compiles research results to date of cementitious and pozzolanic binders as well as filler materials. The following conclusions are made:

Cement is well established as the binder of choice in DSM ground improvement. Lime binders require further secondary reaction with any existing pozzolanic materials in the soil, hence it is not as effective in soils deficient in pozzolans. Strength development progresses, not to the same degree as for cement and take a longer time measured in weeks / months. With cement binders, increasing dosage leads to proportionate increase in treated soil strength gain.

With pozzolans, effectiveness depends on % and form of Si material (more reactive amorphous form vs. crystalline), binder particle surface area / fineness, pH level in the soil, presence of calcium (Ca) (since pozzolanic reactions are also calcium based like cement hydration), and enough quantity and distribution of the Si/Al/Ca components. Aluminosilicate pozzolans are more effective than siliceous pozzolans. MK is identified as the most effect pozzolan when combined with cement. For pozzolans, an optimum dosage limit applies where improvement effects peak (~10 % replacement of cementitious binder being reported).

Although not contributing to further hydration or pozzolanic reaction, filler materials such well-graded silica sand provide solid particles for the reacted binder particle to adhere to. This is applicable to soils with very high water and organic matter content such as peat [67, 84, 89].

Possible research opportunities to pursue have been identified. They include:
- A stoichiometric approach expanding on earlier studies to determine proportions of different pozzolans when combined with cement / lime binders in different soil conditions. Optimum dosage for various pozzolans can then be established for different soils and validated with laboratory testing;
- The effect of different filler materials and proportions to treated soil strength improvement;
- Study into the effect on dynamic properties of binder treated soils;
- The hydration and pozzolanic strength development of different clay mineral types can be tested. By determining the clay content and clay mineralogy of the soil, and therefore understand the pozzolanic content and reactivity, effectiveness with different clay types can be established;
- Determine efficiency of the both hydration and pozzolanic reactions which can be derived by determining Ca (OH)₂ content in treated soil at different curing periods. They can also be co-related with other parameters, notably, water/binder ratio (w/b), plasticity index (PI) and void ratio (e);
- Testing and comparison between field and laboratory samples to measure the difference between in reaction effectiveness and efficiency due to mixing and installation;
- Non pozzolanic materials – e.g. reinforcement fibers, nanomaterials, and alkali activated materials and geopolymers.

9. Acknowledgements

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10. Conflicts of Interest

The authors declare no conflict of interest.

11. References


