Machine Learning Algorithms Evaluation Methods by Utilizing R

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1. Introduction
To date, various technologies have been developed, especially in artificial intelligence (AI). One of the most common field in AI is Machine Learning (ML). ML is one of the most important components in AI (Smola and Vishwanathan, 2010). The idea of ML is lies in the fact that which a computer program is able to learn and develop that leads to construction of new data without interference by human beings (Mujtaba, 2020; Hamarashid, 2021). Various measurements are included to collect, pre-process, and contribute valuable information from datasets. ML, which is an important feature of AI, is utilized for processing massive data. Various ML applications are designed via complicated algorithms that are built in with computer programs. This program constructs a model that describes the dataset. In
addition, the model utilizes parameters inside the algorithms, in which a decision-making procedure can be designed and constructed. Thus, ML is utilized in various fields, for instance, e-commerce, marketing, etc. which has the capability to provide significant suggestions and recommend users demands accurately relying on their history searches and earlier transactions (Kumar, 2018). Depending on this concept, ML could be involved to predict stock price, loan, etc. Besides, it could be used in fraud detection for bank systems. Therefore, ML could be incorporated in a broader area including business, government sectors, etc.

There are various types of ML algorithms, which are supervised, unsupervised, semi-supervised and reinforcement categories.

1.1. Supervised ML Algorithms
In this type, the algorithm could be applied to what has learned from experience to a new data by utilizing a labelled class, for instance, predict future events. This could start by analysing trained dataset. Then, the ML algorithm derives a method to construct predictions on the outcome values. Besides, after adequate training, it has the ability to supply a target to a new input. In addition, the algorithm has the capability to compare obtained results with the correct, expected results and discover mistakes or errors to alter the model appropriately (Expert System Team, 2020; Hamarashid et al, 2021).

1.2. Unsupervised ML Algorithms
This type of ML algorithms is utilized in a situation where the utilized data in training set is not classified or labelled. Unsupervised ML deals with the systems that are able to conclude a method to explain a hidden part from unlabelled data. In this type of ML, the system does not appraise the correct result, but it discovers the data which enables the deduction or assumption to be picked from the data to illustrate the hidden part or feature from unlabelled data (Hamarashid et al, 2021).

1.3. Semi-Supervised ML Algorithms
This category could be in between Supervised and Unsupervised ML types, because Semi-Supervised could utilize both Supervised and Unsupervised. In other words, this means labelled and unlabelled learning during training data, especially utilizing a small or limited amount of data in labelled class, in addition to a huge amount of data in unlabelled data. In this type of ML, learning accuracy could be significantly improved during designing the systems. Put it another way, these types of systems have the ability to increase the accuracy of learning significantly. In Semi-Supervised learning, to achieve or obtain unlabelled data of relevant and associated resources are not basically required. In contrast, to obtain labelled data, associated resources and additional resources are required. This is conducted in order to train/test data and learn from the data (Reddy et al, 2018; Van Engelen and Hoos, 2020).

1.4. Reinforcement ML Algorithms
This type of ML has a connection or related with its environment by construction functions and discovering rewards or errors. Characteristics of reinforcement learning contains delayed rewards and searching for trial and errors. In this technique, the machines are permitted to discover the optimal behaviour with determined content; the purpose is to maximize performance. To learn which function or method is the optimal/best value, the reward feedback is required for the machine (François-Lavet et al, 2018; Mahesh, 2020).

Figure 1 shows types of machine learning algorithms.
One measure for assessing the techniques is accuracy. Accuracy refers to the percentage of correct predictions made by the technique or model. The following is the formal definition of accuracy:

\[
\text{Accuracy} = \frac{\text{Number of correct predictions}}{\text{total number of predictions}}
\]  

On the other hand, accuracy is calculated based on the following formula to calculate accuracy in terms of positives and negatives for binary data and classification:

\[
\text{Accuracy} = \frac{TP+TN}{TP+TN+FP+FN}
\]

Where TP stands for True Positives, TN stands for True Negatives, FP stands for False Positives, and FN stands for False Negatives.

This research paper consists of various features that are addressed in the following sections. The rest of the paper is concerned with determining or selecting appropriate algorithm for a problem. After that, utilizing trial and error to select an algorithm is discussed. Then, Spot-check algorithm is illustrated. Later, test options, building models and selecting models are represented respectively.

2. Methodology

Selecting an appropriate algorithm for a problem is an important issue by researchers. All the researchers seek the best and the most appropriate model for their dataset. This is the main aim of all researchers during their studies, especially for the predictive models. On the other hand, there is no researcher can determine which model is suitable or the best for a provided or determined dataset for the purpose of getting the best outcome. However, sometimes because of having a deep knowledge for a difficulty by the researchers, they know which algorithm could give the best outcome only for a specific dataset. At this time, ML is not the first place or choice. Otherwise, the researchers do not know an appropriate learning algorithm to be utilized for solving a problem. In addition, they do not know which parameters are the best to be utilized with an algorithm to solve a problem. There are different ways to support handling this difficulty. Figure 2 represents the research paper methodology to select the most appropriate model.
2.1. Utilize Past Experience
Experience is an option that researchers can depend on to choose an algorithm for handling a problem. This can include the research works that have previously examined similar problems. It might include several experiences collected in a determined field by conducting literature review on the related research papers, books, and other resources to find out
the idea of which algorithms fits or obtain the best results considering the difficulty arisen. This is an important point to start but not the point to stop on. Thus, trial and error finding could be utilized.

2.2. Utilize Trial and Error
Utilizing trial and error is another method that could be used to discover a better algorithm or appropriate algorithm for a provided dataset. Accordingly, various set of algorithms on the provided dataset is assessed to discover which algorithm is appropriate and what algorithm is not appropriate for the determined dataset. This can be named as spot-check for the algorithms. To conduct this task, a brief list of algorithms is available as a suitable algorithm for the determined problem, and the researcher should be good at selecting the appropriate algorithm to figure out the problem. Therefore, the brief list of algorithms should be concentrated on to choose the best and most appropriate algorithm. Moreover, these algorithms could be improved by altering the algorithm parameters or tuning them. On the other hand, the algorithms could be developed by hybridizing multiple algorithms such as mixing predictive algorithms, and ensemble techniques could be utilized.

3. Algorithms spot-check
This part represents a case study to assess appropriate algorithms for a testing purpose by utilizing R Programming. The utilized test problem in this case study contains a class variable to classify the dataset. The dataset is about diabetes on female patients. The dataset consists of bio-information records for the patients with the class variable. The class variable records are binary or Boolean value. The class variable results reveals that the patient has started diabetes or not, based on earlier medical assessment. Three phases can be conducted to do the task; test on the dataset, then, creating multi-prediction model depending on the data; After that, techniques are compared to choose a brief list of algorithms.

3.1. Testing Data
The first step is about testing the data that includes three points: first, the dataset that is utilized for training the model; second, the utilized test options to assess the technique like re-sampling technique; last, the fascinating metrics to measure and compare.

In the utilized data, the problem should be represented to spot-check algorithm. Besides, the entire data should not be included. The task of spot-checking algorithms must be quick. If there is a huge amount of data, then to train it, more time is needed to compute with the algorithm that desired to check. Checking or training is different from a dataset to another, but it is recommended to check, for example, 15000 instances. This may take few minutes to process. In contrast, for a massive amount of data, a random instance from the huge dataset should be taken. Thus, it can be found out later that the taken sample instances affect a brief list of selected algorithms. Therefore, to load the data to the program the following steps was done:

```r
# import data
data(biomedicaldata)
biodata <- biomedicaldata
```

3.2. Test Preferences
After the data is loaded, test preferences should be considered. Test choices are associated with the algorithms that are utilized to conduct the unseen data accuracy in a model. Re-sampling technique in statistics is an example. Test choices are as follows:

1. Splitting train and test data: if the data is huge, then a massive amount of data is needed to make the model accurate.
2. Cross validation: a frequently utilized agreement of pace, computation time, and generalize error assessment by using 5 to 10 folds.
3. Cross validation repeat: if a small amount of data is provided, by utilizing 5 to 10 folds and repeating it, three times to provide more strong estimation. To conduct this task, the following code snippet was used:
A random number of seed variable was set in order to be re-set before each algorithm was trained. It is crucial to be assure that every algorithm was assessed on the same split or separated data. Later, this process permits true to true comparison rigorously.

### 3.3. Test Metrics
The test metric was used to evaluate the model. There are various available metrics to be selected to evaluate the model. Several significant test metrics that can be utilized in various difficulty types are as follows:

- **Kappa**: it is easily understandable which takes the class distribution as a base.
  
  \[
  Kappa = \frac{p_0 - p_e}{1 - p_e}
  \]  
  \[\text{(3)}\]
  
  \[
  p_0 = \frac{\text{number in agreement}}{\text{total}}
  \]  
  \[\text{(4)}\]
  
  \[
  p_e = P_{\text{correct}} + P_{\text{incorrect}}
  \]  
  \[\text{(5)}\]

- **Accuracy**: correct prediction is divided by total prediction. This is the most common metric that is utilized to assess a model.
  
  \[
  \text{Accuracy} = \frac{x_{\text{correct}}}{y_{\text{total instance}}}
  \]  
  \[\text{(6)}\]

- **Goodness of fit**: It is determination coefficient.

- **RMSE**: Root mean square error is another common metric, which depends on discovering mean square error then the root of MSE will provide the RMSE value.
  
  \[
  \text{RMSE} = \sqrt{\text{MSE}}
  \]  
  \[\text{(7)}\]
  
  \[
  \text{RMSE} = \frac{\sum_{i=1}^{n} (x_i - \hat{x}_i)^2}{n}
  \]  
  \[\text{(8)}\]

Therefore, the following code snippet reveals how to determine the metric:

```r
seed <- 5
ctl <- trainCtl(method="repeatedcv", number=10, repeats=3)
```

### 4. Constructing Models
To spot-check algorithms during denominating methods, three points should be taken into consideration. First, which methods should be selected? Second, how to arrange and construct methods parameters? Last, data pre-processing for the algorithm.

According to algorithms, it is crucial to have a combination of algorithms. There are different methods that can be utilized as shown below:

- **Linear algorithm**: It is used with logistic regression and linear inequity analysis.
- **Non-linear algorithm**: It includes Naïve Bayes, support vector machine, K-nearest neighbour, and neural network, etc. algorithms.
- **Rules and Tree**: PART, J48, and CART are examples algorithms.
- **Tree ensembles**: stochastic gradient boosting, random forest, C5.0, and bagged CART algorithms.
The complicatedness of algorithms is different. If the algorithms with less sophisticated is desired, then KNN as an example is a choice. Besides, the algorithm can be developed. On the other hand, if an algorithm with more complicatedness is desired, then random forest RF is a choice to figure out if the difficulty could be solved and also to start making accuracy anticipations.

For configuring algorithms, the entire ML algorithms have nearly parameterized. Thus, the algorithms parameters argument are required to be specified. In addition, the algorithms that are heuristic can be utilized to provide the first past algorithm configuration to begin. Therefore, during spot-checking, it is not appropriate to attempt different parameters in algorithms which appear during the developing consequences. To do this, R package under name Caret is better to be utilized because it supports tuning parameters of algorithms.

Data pre-processing is another point that several algorithms execute the entire data pre-processing instead of performing fundamental pre-processing. To provide equality in chances of selecting algorithms, it is vitally important that necessary pre-processing is included in the training data for those algorithms that are mandatory to be conducted. For instance, those algorithms that depend on many instances perform better, where the entire input attribute have the same scale. To conduct pre-processing, the train () method inside Caret package was used. This allows to determine pre-processing the data to conduct preceding to training. The necessary changes are provided to the pre-processing parameters argument list, and they are performed sequentially on the data according to (Wu et al, 2008). These algorithms are the most popular algorithms to be conducted in data mining and pre-processing.

1. C4.5: it is a decision tree algorithm that contains ancestor methods for instance C5.0 and ID3 algorithms.
2. K-Means: it is a clustering algorithm.
3. SVM: it is mostly utilized in classification.
4. Apriori: It is used for rule extraction.
5. EM: It is a clustering algorithm.
6. AdaBoost: it is for ensemble methods.
7. KNN: it is an effective technique that is instance-based technique.
8. Naive Bayes: it utilizes Bayes theorem.
9. CART: it is working on trees of classification and regression.

This can be done by utilizing the following code snippet:

```r
Pre-Processing=c("center", "scale")
```

To execute spot-checking for algorithms, the following code snippet is performed:
5. Selecting Appropriate Model

After the data has been trained with various algorithms, they are necessary to be assessed and compared. At this step, the best algorithm is not searched for because they are not tuned which provides better results than the current results. The main aim is to select the well performed algorithm. To perform this task, the following code snippet was conducted:

```r
# Linear Analysis
set.seed(seed)
fit.lda <- train(biomedicaldata ~., data=biodata, method="lda", metric=metric, preProc=c("center", "scale"), trControl=control)

# Log Regression
set.seed(seed)
fit.glm <- train(biomedicaldata ~., data=biodata, method="glm", metric=metric, trControl=control)

# GLMNET
set.seed(seed)
fit.glmnet <- train(biomedicaldata ~., data=biodata, method="glmnet", metric=metric, preProc=c("center", "scale"), trControl=control)

# SVM
set.seed(seed)
fit.svmRadial <- train(biomedicaldata ~., data=biodata, method="svmRadial", metric=metric, preProcessing=c("center", "scale"), trControl=control, fit=FALSE)

# KNN
set.seed(seed)
fit.knn <- train(biomedicaldata ~., data=biodata, method="knn", metric=metric, preProc=c("center", "scale"), trControl=control)

# Naive Bayes
set.seed(seed)
fit.nb <- train(biomedicaldata ~., data=biodata, method="nb", metric=metric, trControl=control)

# CART
set.seed(seed)
fit.cart <- train(biomedicaldata ~., data=biodata, method="rpart", metric=metric, trControl=control)

# C5.0
set.seed(seed)
fit.c50 <- train(biomedicaldata ~., data=biodata, method="C5.0", metric=metric, trControl=control)

# Bagged CART
set.seed(seed)
fit.treebag <- train(biomedicaldata ~., data=biodata, method="treebag", metric=metric, trControl=control)

# Random Forest
set.seed(seed)
fit.rf <- train(biomedicaldata ~., data=biodata, method="rf", metric=metric, trControl=control)

# Stochastic Gradient Boosting
set.seed(seed)
fit.gbm <- train(biomedicaldata ~., data=biodata, method="gbm", metric=metric, trControl=control, verbose=FALSE)
```
The summary of the consequences of accuracy is shown in Table 1.

<table>
<thead>
<tr>
<th>Algorithm</th>
<th>Average</th>
<th>Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>LDA</strong></td>
<td>0.775</td>
<td>0.870</td>
</tr>
<tr>
<td><strong>Logistic</strong></td>
<td>0.778</td>
<td>0.870</td>
</tr>
<tr>
<td><strong>Glmnet</strong></td>
<td>0.777</td>
<td>0.870</td>
</tr>
<tr>
<td><strong>SVM</strong></td>
<td>0.765</td>
<td>0.896</td>
</tr>
<tr>
<td><strong>KNN</strong></td>
<td>0.746</td>
<td>0.896</td>
</tr>
<tr>
<td><strong>NB</strong></td>
<td>0.756</td>
<td>0.857</td>
</tr>
<tr>
<td><strong>CART</strong></td>
<td>0.738</td>
<td>0.844</td>
</tr>
<tr>
<td><strong>C50</strong></td>
<td>0.758</td>
<td>0.883</td>
</tr>
<tr>
<td><strong>Bagging</strong></td>
<td>0.753</td>
<td>0.857</td>
</tr>
<tr>
<td><strong>RF</strong></td>
<td>0.761</td>
<td>0.857</td>
</tr>
<tr>
<td><strong>GBM</strong></td>
<td>0.770</td>
<td>0.883</td>
</tr>
</tbody>
</table>

According to Table 1, the outcome shows that linear algorithms, such as LDA, Log, Glmnet and GBM perform better to be conducted on biomedical data on diabetes for women. Figure 3 shows the consequences of algorithms accuracy:
5.1 Spot-checking for Better Algorithm

In this research paper, several recommendations were addressed to select or spot a significant algorithm. These following suggestions are supportive at assessing machine-learning algorithms by utilizing R programming:

1. Pace or speed: It is important to obtain consequences fast, utilize small amount of sample data and utilize basic anticipates for parameters of the algorithm. This might take a minute to an hour depending on the sample dataset.

2. Variety: It is useful to utilize various algorithms for the representations. Besides, for the same representation, utilize various learning algorithms.

3. Scale-Up: Researchers could follow scaling up with spot-checking algorithms for greater amount of sample data. Nevertheless, this needs more time. Besides, it might need a more powerful computer, but it helps to obtain algorithms that perform better with greater amount of sample data.

4. A brief list: One of the purposes of spot-checking algorithms is to make a brief list of algorithms to find out more, not optimal accuracy.

5. Heuristic: It is the best form to experience algorithms configuration.

6. Conclusion

In conclusion, this research paper helps to find out the significance of spot-checking machine-learning ML algorithms for a problem. In addition, it reveals that the spot-checking is the best option to discover a good ML algorithm for a provided dataset. A case study on diabetes patient for women dataset was considered by utilizing R programming. Besides, various algorithms were assessed on a class variable which is a binary class variable. Moreover, a better algorithm has been selected based on the tests conducted and the results obtained and compared. Consequently, this work answers the questions: which algorithm should be utilized on a dataset and how to investigate ML algorithm. We have noticed that in recent years, machine learning algorithms are still considered one of the most promising models in terms of being integrated with other technologies. This means that the changes and developments in it are continuing to come up with new methods that can be more effective and capable of yielding more satisfying results. For future reading, the authors advise the reader could optionally read the following research works (Hassan and Rashid, 2019, 2020, 2021; Hassan, 2020; Hassan et al, 2016, 2021a, 2021b).
References
Aspect Oriented Programming: Trends and Applications

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Abstract

The competitive advantage of aspect oriented programming (AOP) is that it improves the maintainability and understandability of software systems by modularizing crosscutting concerns. However, some concerns, such as logging or debugging, may be overlooked and should be entangled and distributed across the code base. AOP is a software development paradigm that enables developers to capture crosscutting concerns in split-aspect modes. Additionally, it is a novel notion that has the potential to improve the quality of software programs by removing the complexity involved with the production of code tangles via the usage of separation of concerns. As a result, it provides more modularity. Throughout its early development, some believed that AOP was easier to build and maintain than other implementations since it was based on an existing one. The statements are predicated on the premise that local improvements are easier to implement. Additionally, without appropriate visualization tools for both static and dynamic structures, crosscutting challenges may be difficult for developers and researchers to appreciate. In recent years, AspectJ has begun to enable the depiction of crosscutting concerns via the release of IDE plugins. This article explains aspect oriented programming and how it may be used to improve the readability and maintainability of software projects. Additionally, it will evaluate the challenges it presents to application developers and academics.

Keywords: Aspect Oriented Programming, AOP, Logging, Cross Cutting Concerns, Join Points, Point Cut.

1. Introduction and Background

Since the inception of programming languages, a multitude of programming paradigms have been established in response to technological advancements. Nowadays, the complexity of software increased, manufacturers were forced to broaden and adapt their methods to emerging difficulties. As a consequence of the emergence of programming languages, prototypes of procedural and object-oriented programming (OOP) have been built. Of the ongoing innovation that has happened in order for them to stay competitive. In recent years, OOP has established itself as the primary technique for writing successful structure-oriented code. In comparison to OOP, AOP is a relatively recent technique that has made substantial advancements (Zhang, 2011). According to (Rademacher et al., 2019), their study demonstrates that OOP may
fundamentally aid software engineering by providing a better match for the programming challenge through the core object model. However, AOP complements OOP by introducing a novel approach to program structuring. Apart from classes, it provides features that facilitate the modularization of concerns. For instance, transaction management can be applied to a variety of different kinds of objects (Lau et al., 2018). Additionally, it is an innovative software design and implementation approach proposed by Xerox PARC experts (Lemos et al., 2006). Furthermore, the AOP divides the process into two components: the base program and the aspect program.

The base code contains the essential aspects of the system, and object-oriented programming may be implemented. Additionally, the aspect program incorporates cross-cutting characteristics via the use of modularization in addition to the basic software programming (Beneken et al., 2005). The latest generation of AOP technology is distinct from technology that is more organized for more effective design and coding. This division may be necessary to show the creators' desire to think about it. Additionally, one of the benefits of AOP is that it is built on current technologies and may provide additional mechanisms for influencing the development of systems in a cross-cutting issue manner. One aspect is capable of assisting in the deployment of a range of activities, components, or objects. It might be homogenous or heterogeneous. For example, while leveraging the homogeneous feature, logging behavior should take specific processes into consideration. By contrast, heterogeneity may be used to implement both sides of a protocol between two distinct classes.

In contrast to previous separation of concerns approaches, AOP's primary goal is to build more modular architectures and code; the majority of concerns are localized rather than distributed. A well-defined interface enables reasoning about a range of issues in relative isolation. Thus, they become (un)pluggable and customizable via independent development (Alshareef et al., 2020). This article will provide an introduction of aspect-oriented programming and explain the notion of a common set of important terminology. Additionally, the possibility for increased software application comprehension and maintainability will be highlighted. Additionally, it will assess the difficulties it poses for both application developers and academics. Finally, a comparison between AOP with OOP in terms of software maintainability will be made.

2. Related Works
AOP utilizes many of the following terminologies, and each of these terms need to be explained in order to have a fundamental comprehensive of the idea of AOP. However, it should be noted that individual implementations and AOP the existance of varities frameworks.

2.3. Cross Cutting Concern
According to (Rajan and Sullivan, 2005), a cross-cutting issue is a dimension within which design decisions are made. When it is recognized in conventional object-oriented designs, it becomes cross-cutting and may result in dispersed (code duplication) or tangled code (significant interdependence across systems), or both. As a result of code dispersion, cross-cutting problems are executed inside the underlying program. For instance, log writing capability is needed in the majority of components. As a result, implementation should spread among them. Additionally, since they are compelled to deal with non-core operations, the responsibilities of the various components will remain unclear. Specifically, the command of components in relation to the number of new features is cross-cutting functionality, sometimes referred to as knotted code. The following Figure 1 illustrates a cross-cutting topic in further detail.
2.4. Aspect
Aspect is a modular unit that satisfies cross-cutting criteria and may be specified as a class, for example, build. Additionally, it has a number of pointers that are well related. The primary concept of aspect-oriented programming (AOP) is to encapsulate cross-cutting concerns functionally separate from the fundamental core programming in order to differentiate specified modules. For instance, the AOP aspect may be used to identify a logging module. Meanwhile, applications might have a variety of aspects, depending on the aspect language's needs. However, aspects may be built hierarchically, and the language may provide methods for expressing an aspect and associating it with a core system.

2.5. Join Point
A join point for the request's execution is a well-defined place in a program that handles cross-cutting. During execution, instructions referred to as join points may be performed. It varies according to the aspect language; for example, joint point might refer to the methods that are executed, such as exception handler execution or modifying the class's properties (Wand et al., 2001). Typically, each problem has a number of common points. However, if there are just a handful, they might easily modify the code manually. AspectJ is an aspect-oriented modification to Java that has been used as an example to aid in comprehension; while constructing an aspect, join points are specified. When specifying point cuts, it must determine which joint point performs the action. Additionally, it has a limited number of accessible join points for doing the following: (Vidal et al., 2015).

1. Calling methods
2. Calling constructor
3. Writing or reading access to a filed
4. Initializing execution of class and objects
5. Executing exception handler

2.6. Point Cut
A point cut is a collection of one or more entry points used to access the running program's execution. One of the points indicated in the pointcut is also used by guidance and should be implemented. A programmer may specify the time and

Figure 1. Cross Cutting Concerns (Ju and Bo, 2007)
location of when and where the additional code shall be executed (Avgustinov et al., 2007). AspectJ’s point cut expression is one of the most well-known varieties of point cut expression languages. The model’s appearance is shown in Figure 2.

```java
point_cut aspectJOnPC ()
{
    call_below (call_function (* coloringClient.*(.) & & this (SomeCallerFunction)) & & call (FigureElement Figure.make*(.))
}
```

Figure 2. (sample code) Models of AspectJ pointcut (Lehmann et al., 2012)

2.7. Advice
The advise develops an AOP and functional programming community of functions that change other functions. Additionally, it is a guaranteed function, method, or procedure that is implemented at the point of connection in order to facilitate the execution of a common application (Hentunen, 2015). Additionally, it addresses the functional needs for addressing cross-cutting usability issues. Numerous types of advice are provided in conjunction with various modes of communication. It is decided by the action that advise is capable of calling with the target method around a join point, either before, after, or both.

2.8. Weaving
Weaving is a method for sending suitable advice at each execution step or for linking bits of an advised item to other application requests or objects (Hentunen, 2015). These operations occur during the compilation phase, the load phase, or the run phase (Chapman et al., 2013). An isolated aspect compiler is used to weave the code during the compilation process. For instance, AspectJ is one of the most common compile-time aspect language implementations. The class loader is responsible for weaving throughout the virtual machine class loading process in load-time weaving. Additionally, runtime weaving accomplishes the binding via the usage of proxy classes and the building of code libraries. Furthermore, the Spring Framework is a well-known example of a runtime implementation (Nguyen, 2018).

3. Advantages and Shortcomings of AOP
Utilizing aspect-oriented programming has a number of benefits. For instance, it may be an effective method for resolving cross-cutting issues. However, various disadvantages may develop.

3.1 Advantages of AOP
As previously stated, AOP enables the management of cross-cutting business functions by encapsulating the desired system. Alternatively, the function may be divided into discrete modules. Since a consequence, the entire structure may be compromised, as the base program is not accountable for cross-cutting functionality (Raheman et al., 2018). This technique will automatically result in less code duplication and a lower chance of errors. OOP may also be used to expand or modify the usability of it is feasible that fresh extra features can be implemented without modifying the fundamental software. While this is useful when the source code is unavailable for other reasons, it must be deleted since it may result in altering side effects. Additionally, it may be used to evaluate apps through software testing guidance may be necessary to invoke methods and track application counts and execution time. Advice may be validated prior to and after the use of the approaches. Refer to Figure 3.

```java
before () : examplePoint_cut()
{
    //executable code goes here
}

after () returning : examplePoint_cut()
{
    //executable code goes here
}
```

Figure 3. (Sample code) Simple example of creating advice (before and after) (Gulia et al., 2019)
3 Disadvantages of AOP

AOP is a very efficient programming approach that is dependent on the implementation of the aspect language. This procedure may involve the addition of class attribute values that may be used to adjust methods' arguments and return values. Additionally, this might open up new options that can result in an increase in the system's process complexity, resulting in difficult-to-trace faults. According to (Raheman et al., 2018), the use of AOP may add to the system's complexity. This is reason for a programmer to explore any associated classes and characteristics in order to comprehend the system's behavior. Additionally, when applied to AOP, the usability of the base program and any aspect programs that are necessary for comprehending the program in its whole. Additionally, it is necessary to study the characteristics of the super class in order to comprehend its subclass. Additionally, one potential disadvantage arises in the point cuts that connect the join points with the others' advise. For instance, the description of the error point cut may result in guidance to be compelled to enter incorrect join points or to avoid entering at all stages of the join points. Figure 4 illustrates the many types of mistakes that might occur while describing a point cut.

![Figure 4. Potential Error Situations with Point Cuts.](image)

The following mistake scenarios are possible with point cuts (Lemos et al., 2006); see Figure 4:
1. Point cut selects a subset of the join points offered.
2. No join points were identified by the point cut.
3. A point cut selects anything that is desired or incidental.
4. Point cut deliberately selects many, but not all.

Inaccuracies in the thumbnail description may result in difficult-to-detect behavior. If the system has connected pointcut faults, the effective implementation of an aspect-based transaction management technique may result in a number of system difficulties.

4. Evaluating AOP against OOP

The comparison of AOP vs OOP focuses mostly on cross-cutting concerns. AOP is used to modularize dispersed code; creating modules from scattered code simplifies logging and improves the readability of the code. In contrast to AOP, OOP cannot be represented physically. However, AOP is a seldom utilized technique that may incur considerable runtime costs.
There are two methods in Figure 5, 'addOne' and 'subtractOne', as well as two log calls that have no influence on the functionality of the calling method. These techniques may need extensive and sophisticated code as a result of the implementation of this kind of cross-cutting issue; one advantage is that the logging may be abstracted through class functions. Additionally, the log aspect abstracts whole logs. Demonstrates how to use the logger. By comparison, Figure 6 Additionally, 'MyClass' methods have a unique and suitable functionality where all handling is contained inside a single function.
5. AspectJ in AOP
AOP’s primary goal is to try to resolve code tangling and scattering difficulties by modularizing and encapsulating crosscutting concerns. It introduces a novel concept referred to as an element in order to include a variety of cross-cutting problems (Mcheick and Godmaire, 2018). Additionally, AspectJ may communicate with the underlying code through pointers, advice, and type declarations. Refer to Figure 7, which has an example of Java code.

```java
aspect TracingExample {
    // capture the execution of methods in
    // classes named * in methods named *
    // with any parameters in package
    // "example"

    point_cut trace () : execution (* * (.)) && within (example);

    before () : trace () {
        // Excute this code
        // before the above point is reached
        System.out.println("Entering\nthisJoinPoint.getSignature().getName()\n");
    }
}
```

Figure 7. (Sample code) AspectJ code snippet (Lehmann et al., 2012)

In terms of intertype declarations or intro-durations, they are a method that enables application developers to crosscut concerns in a static manner. For instance, extending a class with additional methods or characteristics (Panwar et al., 2019).

6. Concluding Remarks
Through the use of AOP, a novel and inventive technique for resolving cross-cutting issues has arisen. AOP provides an alternate answer to issues that are difficult to handle with conventional OOP. However, as the system evolves and becomes more complicated, AOP has yet to make a fundamental breakthrough that will answer all cross-cutting issues in the near future. At the moment, AOP is unable to function successfully due to a lack of developing materials and resource information from other initiatives. Despite these difficulties, it is expected to be included into future programming standards. AOP is used to develop and construct dynamic and structured applications. AOP paired with AOP may create a novel approach for programming in which the AOP may provide tools for managing cross-cutting difficulties. As a result, the combination of object and aspect orientation is advantageous. The modularized nature of cross-cutting activities may result in a succinct and ordered structure, which can assist decrease the likelihood of mistake. This is made feasible by the ability to incorporate the functionality of several components into software. As a consequence, less code is required than would be necessary with pure OOP software. Additionally, AOP is more effective. Additionally, to what has been said, despite the many benefits of AOP, there are some worries concerning its transparent and nonintrusive functions. For instance, characteristics may be adjusted independently of source code access information using the present essential program features. Additionally, researchers assert that programmers who improve their comprehension and maintainability of application components will get superior outcomes in the era of programming languages. We have noticed that in recent years, AOP is still considered one of the most promising programming languages in terms of being integrated with other technologies. This means that the changes and developments in it are continuing to come up with new methods that can be more effective and capable of yielding more satisfying results. For future reading, the authors advise the reader could optionally read the following research works (Bryar A.Hassan, 2020; B. A. Hassan, 2020, 2021; B. A. Hassan, Ahmed, Saeed, and Saeed, 2016; B. A. Hassan and Qader, 2021; B. A. Hassan and Rashid, 2019, 2021a; B. A. Hassan, Rashid, and Hamarashid, 2021; B. A. Hassan, Rashid, and Mirjalili, 2021; B. Hassan and Dasmahapatra, n.d.; Maaroof et al., 2022; Saeed, Hassan, and Qader, 2017)(B. A. Hassan and Rashid, 2021b)

References


Applicability of Nanoparticle Flooding Process in a Carbonate Rock of Kurdistan Region: Experimental Investigation of Interfacial Tension and Wettability

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Abstract
Enhanced oil recovery (EOR) has long proven to be a good method to mobilize the residual oil that is by passed and capillary trapped by secondary recovery methods. Chemical EOR methods enhance the microscopic and macroscopic efficiency, and ultimately the overall oil recovery is improved. However, the adsorption rate of the surfactant, low resistance to high temperature and salinity are some of the factors that would turn chemical flooding impractical and uneconomic in many cases. Lately, the application of nanotechnology in enhanced oil recovery has showcased some good and prolific results in terms of incremental oil recovery. In this study, the applicability of Nanoparticle flooding in carbonate rocks of Pilaspi formation was probed through a series of tests such as thin section analysis, x-ray diffraction, x-ray fluorescence, interfacial tension and contact angle measurements. The results showed that the composition of the carbonate rocks is predominantly calcite (CaCO₃) with minor traces of quartz and dolomite. From the interfacial tension (IFT) measurements, it was figured out that the silica and alumina Nanofluids lowered the IFT by 27% and 42% with the light oil, and 43% and 49% with the heavy oil, respectively. The contact angle measurements revealed that the Alumina Nano-fluid at 0.25 wt. % reduced the contact angle on the surface of the light and heavy oil aged thin sections from 169° and 115° to nearly 119° and 78°. On the other hand, the silica nanoparticle at 0.25 wt. % reduced the contact angles on both thin section types to around 129° and 80°, respectively.

Keywords: Interfacial Tension, Wettability, Carbonate Reservoirs, Nano-Fluid Flooding, Enhanced Oil Recovery.

1. Introduction
Petroleum Industry has always been highly expected to meet energy demand as it has predominantly been one of the most common sources of energy for many decades. Primary and secondary recovery methods of petroleum industry have shown to be tremendously limited in producing and draining the reservoirs efficiently. As a result, a huge portion of the oil would usually be capillary trapped or untouched. As the demand for oil increases, the need to develop new recovery techniques and strategies also rises. Enhanced oil recovery (EOR) has proven to be an effective technology that can be properly used to ramp up production. EOR is usually defined as any method that aims to recover the residual and remaining oil that has been trapped and left behind by primary and secondary recovery methods (Donaldson et al, 1985; Mashat et al, 2018). The restrictions imposed on most of chemical flooding projects necessitate the urge to develop and investigate the applicability of other techniques. The implementation of nanotechnology in petroleum industry has been
investigated for nearly over a decade, and it has shown very good and promising results (El-Diasty and Ali, 2015; Nourafkan et al, 2018). Silica Nanoparticles (SiO$_2$ NPs) are the most prevalent nanomaterials utilized in EOR applications. Moreover, they are less costly and can be easily obtained. Nanoparticles (NP) as nanomaterials have a 3D dimension with a size ranging from 1-100 nm possess higher ratio of surface area to volume and therefore have superior thermal, chemical, physical and electrical properties than their bulk counterparts. The most important characteristics of nanoparticles for EOR projects are revolving around large surface to volume ratio (because of their small size) along with charge confinement, chemically altered surfaces, and modification of the structure of the material. Additionally, nanoparticles improve the surfactant’s solution used in EOR applications, lower IFT, change the fluid rheology (mainly when used with polymer), and alter wettability (Marwan and Nageh, 2019). This project aims to experimentally investigate the performance of nanoparticle flooding in carbonate reservoirs of Pilaspi Formation in Kurdistan Region. The two mechanisms of wettability alteration and IFT reduction are going to be investigated to understand the effectiveness of nanoparticles (alumina and silica) as EOR agents.

1.1 Study Area
The study area of this work is around Pirmam that is in Northeast of Erbil. The latitude and longitude of the area are 36°21'39.35" N and 44°11'12.30" E respectively. The rock samples were collected from this area, and the location of the collection can be seen through a satellite image in Figure 1. This project aims to experimentally investigate the performance of nanoparticle flooding in carbonate reservoirs of Pilaspi Formation in Kurdistan Region. The two mechanisms of wettability alteration and IFT reduction are going to be investigated to understand the effectiveness of nanoparticles (alumina and silica) as EOR agents.

![Figure 1. Satellite Image of Pirmam Area (ESSRI, 2013).](image)

1.2 Literature Review
Various nanoparticles have already been utilized as EOR agents to understand how efficient they can be in modifying wettability towards water-wet state and reducing interfacial tension (IFT). Onyekonwu and Ogolo (2010) targeted the ability of three polysilicon Nanoparticles (PSNP) to enhance oil recovery in water wet formations. The NPs used were hydrophobic and lipophilic PSNP (HLPN), lipophobic and hydrophilic PSNP (LHPN), and neutrally wet PSNP (NWPN) while the dispersing phases used were brine and ethanol. The results achieved showed that HLPN and NWPN dispersed in ethanol can be considered as good EOR agents in water wet formations since they improved the recovery. Moreover, Ogolo et al (2012) selected nine nanoparticles (oxides of Aluminum, Zinc, Tin, Iron, magnesium, Nickel, Zirconium, Silane treated Silicon Oxide and Hydrophobic Silicon Oxide) with three different dispersing fluids (distilled water, brine, and ethanol) and investigated their impact on oil recovery. They concluded that the Aluminum Oxide and Silicon Oxides resulted in a very good increase in oil recovery compared to the usage of the dispersing fluids alone. All the nanoparticles gained a rise in oil recovery once dispersed in distilled water except hydrophobic silicon oxide. In addition, Roustaie et al (2012) compared between the effectiveness of a modified silica nanoparticle in improving oil recovery from intermediate and light oil reservoirs in Iran. They used wettability alteration and interfacial tension (IFT) reduction as the main mechanism to mobilize the additional residual oil. It was observed that the nanoparticles altered the wettability and reduce the IFT to a good extent. Also, Bayat et al (2014) practiced and explained the impacts of three
various metal oxides nanoparticles named aluminum oxide, silicon oxide and titanium oxide on intermediate wet limestone samples for EOR purposes at different temperatures of 26°, 40°, 50° and 60° C. Their results showed that the nanoparticles did a good job in lowering IFT and changing wettability towards water wet at most of the temperature ranges. In this paper, a higher temperature value (80° C) is used to grasp whether the nanoparticles (Alumina and Silica) can impact the IFT and wettability intensively.

2 Materials
In this section, the main materials needed for the research are explained in details. The type of rock, core plugs made from the rocks, the thin sections made from the core plugs, the synthesis of the brine, and the properties of the oil samples used are all vital parts of the materials section.

2.1 Rock Sample Collection and Core Plug Preparation
Rock samples from the outcrop of Pilaspi Formation (Figure 2a and 2b) were collected. Pilaspi formation is one of the carbonate reservoir formations in Kurdistan Region, and all the carbonate rocks used within this paper are from this formation. Then, the samples were taken to the laboratory to prepare core plugs (Figure 2d) that will later be used in the experimentations. Core plug machine (Drill Press) as seen in (Figure 2c) was used to make the core plugs. Later, the plugs were put in an oven for nearly 12 hours at a temperature of 100 C to be dried off.

Figure 2a. The Pilaspi Formation.
Figure 2b. Closer View of the Pilaspi Formation.
Figure 2c. Drill Press (core plugging machine).
Figure 2d. Core Plugs.

Figure 2. a) The Pilaspi Formation, b) Closer View of the Pilaspi Formation, c) Drill Press (core plugging machine), and d) Core Plugs.
2.2 Thin Section Preparation, XRF and XRD
Thin sections (Figure 3a) were made from the core plugs using thin section device. Once the thin sections are made, they are properly trimmed on both sides and polished so that the analysis under microscope will be handy and easy to understand. The thin section process was ended by putting the thin sections into an oven (Figure 3b) for nearly 24 hours under a temperature of 80°C. Additionally, aside from the microscopic thin section analysis, x-ray diffraction (XRD) and x-ray fluorescence (XRF) tests were also conducted so that the composition of the rock would be accurately determined. The XRF test was conducted using Spectro XRF with XEPOS model.

2.3 Brine Synthesis and Oil Samples
The reservoir brine used throughout this study was basically synthesized in the laboratory. Tables 1 and 2 elaborate on the composition of the reservoir brine for the two oil samples used. The concentrations of the salts as indicated in Table 1 and 2 were first found, and all the comprising salts were later added to 1 liter of deionized water. Finally, the jar containing the salts and 1 liter of deionized water was placed on a homogenizer with an RPM of 1000 for 24 hours, and the reservoir brine was obtained. The total dissolved solids (TDS) of the brine of the light oil reservoir were nearly 34461.6 ppm while the one of the heavy oil reservoir was approximately 16178.1 ppm. Table 3 shows some properties of the oil samples. The density of the oil & water and the oil viscosity were measured by pycnometer and MCR300 respectively.

<table>
<thead>
<tr>
<th>Ion</th>
<th>Concentration (mg)</th>
<th>Concentration (mole)</th>
<th>Salt</th>
<th>Concentration (gr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cl</td>
<td>16200</td>
<td>0.456942995</td>
<td>NaCl</td>
<td>14.481</td>
</tr>
<tr>
<td>SO₄</td>
<td>1141</td>
<td>0.011877993</td>
<td>Na₂SO₄</td>
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</tr>
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<td>0.085763293</td>
<td>Na₂S</td>
<td>6.712</td>
</tr>
<tr>
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<td>0.026222286</td>
<td>NaHCO₃</td>
<td>2.184</td>
</tr>
<tr>
<td>Ba</td>
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<td>0.000487887</td>
<td>BaCl₂</td>
<td>0.104</td>
</tr>
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<td>NaCl</td>
<td>14.481</td>
</tr>
<tr>
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<td>0.041178261</td>
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<td>CuSO₄</td>
<td>0.479</td>
</tr>
<tr>
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<td>MgCl₂</td>
<td>1.523</td>
</tr>
<tr>
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<th>Salt</th>
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<tr>
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<td>CaCl₂</td>
<td>4.217</td>
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</tbody>
</table>
Table 3. Oil Samples’ Properties.

<table>
<thead>
<tr>
<th>Properties (Unit)</th>
<th>Light Oil</th>
<th>Heavy Oil</th>
</tr>
</thead>
<tbody>
<tr>
<td>API°</td>
<td>36.6</td>
<td>14.3</td>
</tr>
<tr>
<td>Density (Ambient) (gm/cc)</td>
<td>0.745</td>
<td>0.862</td>
</tr>
<tr>
<td>Density (80°C) (gm/cc)</td>
<td>0.8</td>
<td>0.911</td>
</tr>
<tr>
<td>Viscosity (cp)</td>
<td>9.5</td>
<td>150</td>
</tr>
</tbody>
</table>

2.4 Thin Section Aging
The thin sections made earlier need to be restored to an oil wet state. Once the reservoir brine was synthesized and prepared, it would be poured into aging cells and the thin sections are put & immersed inside and sealed tightly. Initially, the thin sections-filled containers are placed inside the oven for 5 days at a temperature of 80°C until the thin sections are aged with water. Then, the water aged thin sections are placed in another jar filled with the oil samples (heavy and light separately) for nearly 20 days inside the oven at a temperature of 80°C.

3 Experimental Work
The main experimental tests conducted throughout this work are: Interfacial Tension (IFT) measurement and contact angle measurement.

3.1 Interfacial Tension Measurement
The IFTs were measured using high pressure high temperature (HP-HT) pendant drop apparatus- model (VIT-ES20) (Figure 4). Aluminium oxide and silicon dioxide nanoparticles were used throughout this paper. Table 4 shows the two nanoparticles along with some of their properties. The nanoparticles were provided by the same German Company that had also provided the surfactants. The nanoparticle dispersions were made by ultrasound wave maker apparatus (Figure 5). The nanoparticle was placed in a small jar as seen, and 100 cc of formation brine (of both oil reservoirs individually) was added to the jar. The mixture was then placed on the device, and the probe completely went inside the jar, so that enough energy was transferred to the solution. The process was finished once the solution was totally homogenous (no insoluble particles existing). For the IFT measurements, different concentrations of both nanoparticles at 0.1, 0.25, 0.5 and 1 wt. % were added to both synthesized brines, and the IFTs were measured with both heavy and light oil samples at 80°C. The IFT of each Nano-fluid with both oil samples was measured.

Table 4. The Properties of Alumina and Silica Nanoparticles.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Alumina Nanoparticle</th>
<th>Silica Nanoparticle</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight</td>
<td>25 g</td>
<td>25 g</td>
</tr>
<tr>
<td>Purity</td>
<td>99%</td>
<td>99.9%</td>
</tr>
<tr>
<td>APS</td>
<td>20 mm</td>
<td>30 mm</td>
</tr>
<tr>
<td>SSA</td>
<td>&lt;200 m²/g</td>
<td>&gt;150 m²/g</td>
</tr>
<tr>
<td>Appearance</td>
<td>White powder</td>
<td>White powder</td>
</tr>
</tbody>
</table>
3.2 Contact Angle Measurement
To show the ability of the solutions to alter the wettability of the carbonate rock thin sections, contact angle method was used. The high pressure-high temperature (HPHT) pendant drop apparatus that was used for IFT measurement was also used to measure the contact angle between the oil wet thin sections and a drop of the solution. The contact angle between each Nano-fluid and the oil wet thin sections (aged with heavy and light oil) were made at nanoparticle concentrations of 0.1 and 0.25 wt. %. The contact angle measurement at this stage lasted 480 hours at a temperature of 80°C.

4 Results and Discussion
The main results and discussion of this work are broken down into: Thin section, XRF, and XRD results, the wettability alteration by nano-fluids, and IFT reduction by nano-fluids.

4.1 Thin Section, XRF and XRD Results
According to the microscopic analysis of the thin sections (Figure 6a), the studied samples were concluded to be microcrystalline limestone. This rock is part of carbonate rocks’ group, and sparry calcite composes the main constituent of the samples. The crystal size ranges between 10 to 270 microns however the crystal size is predominantly revolving around 50 microns. Moreover, the analysis showed that the sample is of high purity, and free of fossils and detrimental minerals. In addition, very few traces of dolomite can be hardly seen within the specimen.

Additionally, x-ray diffraction (XRD) and x-ray fluorescence (XRF) tests were also conducted so that the composition of the rock samples is accurately determined. As it can be seen from the XRF results (table 5), the rock composition is
mainly calcite by nearly 49.44% followed by very few traces of quartz (SiO$_3$) and a low 4.23% of Mg(CO$_3$)$_2$. It is important to understand that a big portion of the samples was lost on ignition (L.O.I) since it was at a temperature of nearly 750 °C for an hour. Lastly, the composition of the samples was further verified to be mainly calcite through XRD (Figure 6b). It was indicated that the samples are highly composed of calcite by nearly 99%, and the remaining 1% was made of quartz. The calcite section has shown the highest peak in the result. It can be concluded from all three tests that the composition of the core plugs is mainly calcite along with small traces of quartz and dolomite.

Figure 6a. Microscopic Image of Thin Sections (Suramairy et al, 2021).

Figure 6b. XRD Results (Suramairy et al, 2021).

Figure 6. a) Microscopic Image of Thin Sections, and b) XRD Results.
Table 5. XRF Results.

<table>
<thead>
<tr>
<th>Component</th>
<th>%</th>
</tr>
</thead>
<tbody>
<tr>
<td>(Calcite) CaCO₃</td>
<td>49.44</td>
</tr>
<tr>
<td>(Quartz) SiO₂</td>
<td>1.16</td>
</tr>
<tr>
<td>(Dolomite) Mg(CO₃)₂</td>
<td>4.23</td>
</tr>
<tr>
<td>L.O.I</td>
<td>45.17</td>
</tr>
</tbody>
</table>

4.2 The Wettability Alteration by Nano-Fluid

The contact angles of the thin sections after being aged by light and heavy oil samples for 480 hours were measured to be nearly 169° and 115° respectively, and these show that the thin sections are in an oil-wetting state. To alter the wettability towards water-wet, the thin sections were treated by nanoparticles (alumina and silica NP) at a temperature of 80°C.

First, upon the treatment of the light oil aged thin sections with formation water, the contact angle for different time periods and up to 240 hours was measured to be nearly 129° (Figure 7). The treatment only took 240 hours because the contact angle values stabilized at that period. This shows that the synthesized water is not effective in altering the wettability of the thin sections from oil-wet to water wet. Moreover, the contact angle was reduced from nearly 115° to 90° upon treating the heavy oil aged thin sections with formation water.

To investigate the impact of the Nano-fluids on the thin sections’ wettability, two concentrations of both SNP and alumina NP at 0.1 and 0.25 wt. % were used. These two concentrations were selected based on the IFT results of the Nano-fluids. The treated contact angles on the surface of both thin section types were reduced as the concentration increased. Alumina Nano-fluid at 0.25 wt. % reduced the contact angle on the surface of the light and heavy oil aged thin sections from 169° and 115° to nearly 119° and 78° respectively (Figure 8a, 8b, 8c, 8d). On the other hand, the silica NP at 0.25 wt. % reduced the contact angles on both thin section types to around 129° and 80° respectively (Figure 9a, 9b, 9c, and 9d). As it can be viewed from Tables (6 and 7), both Nano-fluids are able to reduce the contact angle, but the alteration is not very sharp, and the reason can be attributed to the effect of ionic strength that would make the alteration process slow. The mechanism of the wettability alteration by the Nano-fluids is strongly related to the adsorption of the nanoparticles on the surface of the thin sections. This would subsequently modify the free surface energy and alters the wettability towards more water-wet (Roustaei et al, 2012).
Figure 8. a) Alumina NP 48 hr (light oil) b) 480 hr (light oil) 119° , c) 48 hr 100° (Heavy Oil), and d) 480 hr 78° (Heavy Oil).

Figure 9a. Silica NP 48 hr (165°) Light Oil
Figure 9b. Silica NP 480 hr (129°) Light Oil
Figure 9c. Silica NP 48 hr (103°) Heavy Oil
Figure 9d. Silica NP 480 hr (80°) Heavy Oil

Figure 9. a) Silica NP 48 hr (165°) (light oil), b) 48 hr 129° (light oil), c) 48 hr 103° (Heavy Oil), and d) 480 hr 80° (Heavy Oil).

Table 6. Contact Angle Results of the Nanofluids (Light Oil Thin Sections).

<table>
<thead>
<tr>
<th></th>
<th>Contact Angle</th>
<th>Contact Angle</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>48hr</td>
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<tr>
<td>Alumina 0.1%</td>
<td>163</td>
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<tr>
<td>Time (hr)</td>
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<tr>
<td>Average C.A.</td>
<td>170</td>
<td>163</td>
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<table>
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<tr>
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<th>Contact Angle</th>
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<tr>
<td></td>
<td>48hr</td>
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<tr>
<td>Alumina 0.25%</td>
<td>168</td>
<td>150</td>
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<td></td>
<td>163</td>
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<tr>
<td>Time (hr)</td>
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<td>48</td>
</tr>
<tr>
<td>Average C.A.</td>
<td>169</td>
<td>166</td>
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</tbody>
</table>

Table 7. Contact Angle Results of the Nanofluids (Heavy Oil Thin Sections).

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<th></th>
<th>Contact Angle</th>
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<td>120hr</td>
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<tr>
<td>Silica 0.1%</td>
<td>154</td>
<td>148</td>
</tr>
<tr>
<td></td>
<td>158</td>
<td>143</td>
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<tr>
<td></td>
<td>156</td>
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<tr>
<td>Time (hr)</td>
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<tr>
<td>Average C.A.</td>
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<td>156</td>
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<table>
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<th></th>
<th>Contact Angle</th>
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<tbody>
<tr>
<td></td>
<td>48hr</td>
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<tr>
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<td>149</td>
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<tr>
<td>Time (hr)</td>
<td>0</td>
<td>48</td>
</tr>
<tr>
<td>Average C.A.</td>
<td>168</td>
<td>165</td>
</tr>
</tbody>
</table>
4.3 IFT Reduction by the Nanofluids

In this part of the experimental work, the IFT between water and oil through Nano-fluids was investigated to perceive how effective the nanofluids can be. Both alumina and silica nanoparticles with various concentrations at 0.1, 0.25, 0.5, and 1 wt. % were prepared, and their impact on IFT was examined with both light and heavy oil samples at a temperature of 80°C and a pressure of 100 psi. The IFT measurement experimentations showed that both Nano-fluids can reduce the IFT, but with a slight difference of the alumina Nano-fluid being better. The results showed that both Nano-fluids at 0.1 wt. % resulted in the biggest IFT reduction with both oil types. Alumina Nano-fluid reduced the IFT between the water and light oil to 7.23 mN/m while the silica Nano-fluid lowered the IFT to 9.13 mN/m (Figure 10a). Moreover, the alumina Nano-fluid–heavy oil system resulted in an IFT of 4.82 mN/m whereas the silica Nano-fluid–heavy oil system obtained an IFT of 5.35 mN/m (Figure 10b). Furthermore, any increase of nanoparticle concentration only increased the IFT (Figure 10a and 10b). The mechanism behind the IFT reduction by the Nano-fluids can be related to the interaction between the water phase/Nano-fluid and oil phase/Nano-fluid. Additionally, the adsorption of the Nano particles onto the interface can be another explanation regarding the mechanism (Roustaei et al, 2012). IFT reduction by Nano-fluids has been proven to be one of the mechanisms that would result in improving oil recovery (Onyekonwu and Ogolo, 2010; Shahrabadi et al, 2012; Zaid et al, 2014; Roustaei et., 2012; Hendraningrat and Torsaeter, 2014).

**Figure 10a.** Various Concentrations of Nano-fluids vs. IFT (Light Oil).
5 Conclusions and Recommendations

In this section, the main concluding points of the research including the most important results along with a recommendation are pinpointed.

5.1 Conclusions

The following are the main concluding points of the experimental investigation of the impact of nanoparticle flooding, as an EOR, on oil recovery in carbonate rock of Pilaspi formation in Kurdistan Region.

1) Microscopic analysis, XRD and XRF results were all nearly in-line with each other and showed that the composition of the rock is predominantly calcite (CaCO3) along with minor traces of quartz and dolomite.

2) The silica Nanofluid at 0.1 wt. % lowered the IFT between the aqueous phase and the light oil to 9.13 mN/m while the IFT between the aqueous phase and the heavy oil was reduced to 5.35 mN/m at the same concentration. However, the alumina Nanofluid at 0.1 wt. % shrunk the IFT between the water and the light/heavy oil to 7.23 mN/m and 4.82 mN/m, respectively.

3) The contact angle of the oil wet thin sections upon treating by the formation water was altered from nearly 169° and 115° to 129° and 90° for the light oil and heavy oil, respectively.

4) Alumina Nano-fluid at 0.25 wt. % reduced the contact angle on the surface of the light and heavy oil aged thin sections from 169° and 115° to nearly 119° and 78° respectively while 0.25 wt. % silica Nano-fluid lowered the contact angles to 129° and 80° for the light and heavy oil samples respectively.

5.2 Recommendations

The following point is recommended for further research in this specific era of EOR in Pilaspi formation carbonate rocks: Investigating the applicability of other Nanoparticles such as the oxides of zinc, nickel, tin, iron, magnesium, and zirconium to understand how imperative they can be as EOR agents in Carbonate rocks of Pilaspi.

References


Onyekonwu, M.O. and Ogolo, N.A. (July 2010). Investigating the Use of Nanoparticles in Enhancing Oil Recovery. Nigeria Annual International Conference and Exhibition, Tinapa - Calabar, Nigeria. DOI: https://doi.org/10.2118/140744-MS.


1. Introduction
In general, a schema in XML represents the specifications of objects and identifies the relationships between the objects. Schema definition languages are the recommendations of the World Wide Web Consortium (W3C) which itself is represented by XML. The language provides facilities to describe the structure of an XML document. It is described as rules that apply to a group of XML documents (Lawton, 2015). The main purpose and most popular use of schema is validation. Different kinds of validation can be performed with different kinds of schema. This means the validation
requirements might be different in different situations. There are many circumstances which requires validating XML documents, for example when testing the output of an application to make sure that the data meets the specified requirements, and when receiving an XML document from an external source (in case of Web Services) the data must be validated before inserting it into a legacy system (Gandhi, 2014). At several levels of processing data, validation could occur. For instance, structural validation makes sure that the structure of XML elements and attributes in an XML document satisfy the identified requirements, however it does not illustrate the textual content of the document. Data validation is another kind of validation process, which ensures that the content of an XML document follows the rules which specify the type of information that should be presented (Gandhi, 2014). This paper focuses on constructing a tool to validate XML documents against XSD in terms of structure and data content validations. The validation process is performed using Java programming language, as java provides useful and easy to use Application Programming Interface (API) for the purpose of validating XML against XSD. Furthermore, the data model of XSD and the differences between XSD and DTD are discussed.

The research starts by studying XML schema abstract data model as an approach to determine how the schema of an XML document and the data elements should be organized. Next, two available and common methods to describe the structure and content of XML documents are; Document Type Definition (DTD) and XML Schema are discussed and compared. Based on the advantages of XML Schema over DTD, XML Schema has been used for the validation process. In section five ‘validation technique’ the proposed validation method is explained. Java API for XML Processing (JAXP) is utilized to develop the validator, and all the necessary steps and phases of the validation process is presented in this section. Finally, in the results and discussion section the research mentions the requirements of an application that should be considered while choosing a validation technique.

2. Materials and Methodology
In this paper, the technique used to study XML validation is implemented in Java programming language. Java supports most of the XML related technologies. The proposed validation technique supports those systems and applications that require the process of validation and parsing XML documents to be separated. Therefore, the chosen Application Programming Interface (API) is JAXP Validation API. This API allows the applications to validate an XML document against an XML Schema Definition (XSD) without having to parse the XML document. The most important procedures of making this validation program are presented with details in section “The Validation Technique”.

3. XML Schema Abstract Data Mode
The information set that is expressed in XML schema components identifies the abstract data model of the XML schema (Geroimenko, 2012). According to W3C one fundamental abstract data model can describe all XML Schemas. That model specifies the structure and the data content of the schemas. However, The XML Schema abstract data model is only conceptual, and it does not impose any specific style or structure on the XML documents or schemas. It only provides a large collection of components and dictates how the components should be linked (McKinnon and McKinnon, 2003). As shown in Table 1, W3C describes several kinds of schema components, which can be classified in three groups, which are Primary components, Secondary components and Helper components (Henry and Gao, 2012). The entire Primary component and some of the Secondary components are discussed in this section.

<table>
<thead>
<tr>
<th>Component Group</th>
<th>Schema Components</th>
</tr>
</thead>
<tbody>
<tr>
<td>Primary</td>
<td>Simple type definitions</td>
</tr>
<tr>
<td></td>
<td>Complex type definitions</td>
</tr>
<tr>
<td></td>
<td>Attribute declarations</td>
</tr>
<tr>
<td></td>
<td>Element declarations</td>
</tr>
<tr>
<td>Secondary</td>
<td>Attribute group definitions</td>
</tr>
<tr>
<td></td>
<td>Identity-constraint definitions</td>
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<tr>
<td></td>
<td>Model group definitions</td>
</tr>
<tr>
<td>Helper</td>
<td>Annotations</td>
</tr>
<tr>
<td></td>
<td>Model groups</td>
</tr>
</tbody>
</table>
The root element of an XML Schema is schema, which is usually defined in the schema namespace. Within a schema construct, elements can be declared using element construct <xsd:element>, attributes are also specified using attribute construct <xsd:attribute> (elements and attributes can be declared using xs or xsd prefix. In practice there is no difference between xs and xsd.). Two type constructs are described in XML Schema language, a simple type and a complex type. Complex type is used in almost all meaningful document structures. An element that contains an attribute, a child element or both make a complex type. Complex types are defined in the element declaration using a combination of <xsd:element> and <xsd:complexType>. In contrast, elements that only contain numbers or character strings with child elements are said to be simple types. To declare a simple type a combination of <xsd:element> and <xsd:simpleType> is used (Henry and Gao, 2012) and (Vohra, 2007).

Generally, Schema models are described in terms of constraints to determine what a specific document or language can contain. XML Schemas describe two kinds of constraints: The first is Content Model Constraints, which define the elements that can occur in an XML document. It also specifies the grammar of the language in the documents. The second constraint is Data Type Constraints. These constraints specify the types of data that the schema recognizes as valid (Henry and Gao, 2012), and (Owen and Boyer, 2011). An example of an XML document (“shiporders.xml”) and its XML Schema (“shiporders.xsd”) are shown in Figure 1 and 2.

```
<?xml version="1.0" encoding="UTF-8"?>
<shiporders orderid="123456"
xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance"
xsi:noNamespaceSchemaLocation="shiporders.xsd">
  <orderperson>Sarah Joe</orderperson>
    <shipInfo>
      <name>Chris Tippman</name>
      <address>abstr 23</address>
      <city>123 Berlin</city>
      <country>Germany</country>
    </shipInfo>
  <item>
    <title>Royal Craft</title>
    <note>Special Edition</note>
    <quantity>1</quantity>
    <price>12.80</price>
  </item>
  <item>
    <title>Show your mind</title>
    <quantity>1</quantity>
    <price>10.90</price>
  </item>
</shiporders>
```

Figure 1. An XML Document “shiporders.xml” (W3Schools, 2021).

In the above XML Document “shiporders” is the root element, the attribute “orderid” is identified in this element. The root element “shiporders” specifies three child elements “orderperson”, “shipInfo” and “item”, these elements also contain other different child elements. The sentence `xmlns:xsi="http://www.w3.org/2001/XMLSchema-instance"` indicates that
an XML parser should validate the XML document based on a schema, and the line
<xs:noNamespaceSchemaLocation="shiporders.xsd"> informs the parser about the schema file (in this example the schema file is “shiporders.xsd”) and the location of the file (in this example it is in the same directory as “shiporders.xml”). Figure 2 shows the schema file, which starts with the standard XML declaration and is also organized in XML format.

```xml
<?xml version="1.0" encoding="UTF-8" ?>
<xs:schema xmlns:xs="http://www.w3.org/2001/XMLSchema">
  <xs:element name="shiporders">
    <xs:complexType>
      <xs:sequence>
        <xs:element name="orderperson" type="xs:string"/>
        <xs:element name="shipto">
          <xs:complexType>
            <xs:sequence>
              <xs:element name="name" type="xs:string"/>
              <xs:element name="address" type="xs:string"/>
              <xs:element name="city" type="xs:string"/>
              <xs:element name="country" type="xs:string"/>
            </xs:sequence>
          </xs:complexType>
        </xs:element>
        <xs:element name="item" maxOccurs="unbounded">
          <xs:complexType>
            <xs:sequence>
              <xs:element name="title" type="xs:string"/>
              <xs:element name="note" type="xs:string" minOccurs="0"/>
              <xs:element name="quantity" type="xs:positiveInteger"/>
              <xs:element name="price" type="xs:decimal"/>
            </xs:sequence>
          </xs:complexType>
        </xs:element>
      </xs:sequence>
    </xs:complexType>
  </xs:element>
</xs:schema>
```

Figure 2. An XML Schema “shiporders.xsd” (W3Schools, 2021).

In the above XML Schema, the root element <xs:schema> defines the attribute xmlns:xs which is associated with the URI "http://www.w3.org/2001/XMLSchema" to specify the namespace.

Next, the root element “shiporders”, its attribute and child elements are defined. This element makes a complex type because it contains attribute and child elements. To display the three child elements (“orderperson”, “shipto”, “item”) of “shiporders” in an ordered sequence, the elements are enclosed between <xs:sequence> elements. The “orderperson” is considered as simple type as it does not contain any attribute or child element. However, “shipto” and “item” elements
are of complex types (Owen and Boyer, 2011). The “maxOccurs” and “minOccurs” attributes determine minimum and maximum possible appearance of an element. The default value of these attributes is 1, and if the occurrence of the element is optional then “minOccurs” is set to have the value 0. Here in the declaration of “item” element the value of “maxOccurs” is “unbound” which means the item element can occur many times. “shiporders” has a required attribute “orderid”, the attribute is declared in the end of the schema.

4. XML Schema versus DTD

Document Type Definition (DTD) and XML Schema are two approaches to organize the structure and define the content of an XML document. Although DTD was founded first, there are some significant differences between the two methods, and XML Schema has considerable advantages over DTD. The main advantage of XML Schema is that they are strongly typed and have the ability to define data types of elements and specify their values, length and define other complex structures. This facility can confirm that the data stored in an XML document is valid. While DTD lacks this ability, it is weakly typed and cannot validate data contents to their data types (Joan, 2011), and (Tidwell, 2008). Another difference which makes XML Schema to be more powerful is the fact that XML Schema is written in XML format. Therefore, XML Schemas can be parsed and manipulated like any XML document. In contrast DTD is defined in SGML (Standard Generalized Markup Language). Accordingly, defining the structure and content of an XML document in DTD requires the need to learn a new language (Joan, 2011). Supporting namespace is another characteristic of XML Schema, it can use a set of namespaces to define and organize the Schema. While on the other hand DTD is not aware of namespaces, instead it defines the building blocks of XML documents using its own set of keywords (Ali, 2012). Considering the advantages of XML Schema over DTD, this study focuses on validating XML documents against XML Schema rather than DTD.

5. The Validation Technique

The process of validating XML documents is often necessary to make sure that any system or application which uses the document receives the correct and expected form of data. The validation tool which is developed for this purpose must confirm that the XML documents adhere to the structures and rules specified by a Schema. This section illustrates the steps of developing an XML Schema validator to validate XML documents against XSD using XML related technologies in Java. The validator could then be used when the process of validation needed to be decoupled from parsing. The Application Programming Interface (API) used to develop the validator is Java API for XML Processing (JAXP). JAXP allows the applications to parse, validate and query XML documents. It can be divided into two groups: The first group contains JAXP SAX and DOM parser APIs. SAX (Simple API for XML) is an event based API, it generates a series of events while parsing documents which are handled by callback methods. While DOM (Document Object Model) is an object based API, it represents the XML document in a tree structure so that the application can use the tree nodes for manipulating and querying the data in the document (Li, 2009). These APIs are suitable to be used in applications, which require parsing and validating the XML documents to be combined. In contrast, the second group contains JAXP Validation API. This API is suitable to be used in applications, it require parsing and validating XML documents to be decoupled (Vohra, 2007). This second group API is used to examine the validation process in this research. To validate XML Documents with JAXP Validation API three steps must be taken. Figure 3 is a flowchart diagram that represents the general steps of the validation process.
First: Instantiating three necessary objects.
The three required objects to be obtained are objects from SchemaFactory, Schema, and Validator classes. To be able to create these objects the javax.xml.validation.* package must be imported first. A Schema object representation is necessary to be able to validate with XML Schema based definition (Vohra, 2007). The Schema object is created from SchemaFactory class as shown below:

```java
SchemaFactory factory = SchemaFactory.newInstance(XMLConstants.W3C_XML_SCHEMA_NS_URI);
Schema schema = factory.newSchema(newFile("XMLSchemaFile.XSD"));
```

The parameter to the newSchema() method is the name of an XML Schema file which the validation depends on to check if an XML document follows the specified rules and structures that are defined in the file. The validator object is then created from the schema object as shown below.

```java
Validator validator = schema.newValidator();
```
Second: Reporting Validation Error.
The validator should be able to handle validation errors. To accomplish this task an ErrorHandler class must be defined. This class extends DefaultHandler as shown in Figure 4. An object from this class is set as an argument to the setErrorHandler() method, which is invoked by the “validator” object created in the first step.

```java
private class ErrorHandlerApp extends DefaultHandler{
    public boolean validationErr= false;
    public SAXParseException parseException= null;
    public void error(SAXParseException exception) throws SAXException{
        validationErr= true;
        parseException= exception;
    }
    public void fatalError(SAXParseException exception) throws SAXException{
        validationErr= true;
        parseException= exception;
    }
    public void warning(SAXParseException exception) throws SAXException{
    }
}
```

Figure 4. The ErrorHandlerApp Class.

Creating an object from this class and using it with the setErrorHandler() method is shown below:

```java
ErrorHandlerApp errorHandlerObject= new ErrorHandlerApp();
validator.setErrorHandler(errorHandlerObject);
```

Third: Validating the XML document.
The last step to complete the validation process is validating the XML document. To process the validation, the XML document does not need to be parsed. Alternatively an StreamSource object is created from the XML document, and the validator objects is used to call the validate() method. This method takes the StreamSource object as an argument.

```java
StreamSource streamsrc = new StreamSource (XMLDocument.xml);
Validator.validate(streamsrc);
```

Lastly, the following lines of code were added to check for errors and show the result of the whole validation process. Figure 5 shows the result of the process, if the XML document is not valid then error messages will be shown, otherwise the program indicates that the document is valid.

```java
if (errorHandlerObject.validationErr == true){
    System.out.println("the XML document is not valid"+
    errorHandlerObject.validationErr+"" +
    errorHandlerObject. SAXParseException.getMessage());
} else{
    System.out.println("the XML document is valid");
}
```

Figure 5. The Result of the Validation Process.

6. Result and Discussion
Selecting an approach to validate an XML document depends on satisfying the additional functionality of the validation application. For instance, if parsing an XML document requires being associated with validating the document with a schema, then SAX parser is recommended. However, if the entire tree structure of an XML document needs to be accessed and modified repeatedly, then DOM parser is recommended (Nagrare, 2020b). Furthermore, the parsing process can be performed along with validation if the chosen XML parser can implement validation too. Nevertheless, sometimes the parsing and validation process is required to be separated. The XML validation method examined in this research satisfies this requirement. Separating validation from parsing using JAXP Validation API could be a practical solution for the applications that need to validate an XML document that is not supported by the accessible parser. In addition, with JAXP Validation API, an object is instantiated to represent a schema. This object can be used to validate multiple XML documents. According to (Vohra, 2007) using a single object to validate multiple XML documents is an efficient process. Therefore, JAXP Validation API could be an appropriate method to utilize if for any reason the parsing process of an XML document requires being decoupled from the validation process.

7. Conclusion
With increasing the number of XML related technologies, XML is becoming more popular and is used in a large number of various applications and systems. Therefore, it is vitally important to organize the XML formatted documents in well formed and validated structure and content. This way the systems and applications will be able to predict which kind of data format is received as input, and which structure of data will be produced as output. DTD and XSD are two available standards to define structure and content of XML documents and validate the XML documents against described structures. This study mainly focused on using XSD to accomplish the process of validation, as XSD has more advantages than DTD. For example, XSD has the same structure as XML documents and it is parsed and processed the same way as XML. Furthermore, XSD supports multiple namespaces, and it also has the ability to specify the elements data types and their values to make the validation a reliable process. There are various programming languages that provide techniques to implement the XML validation process. The validation system proposed in this study was implemented in Java Programming language. JAXP is the API which is used for this purpose. This API could be divided into two classes. The first class includes JAXP SAX and DOM, which perform XML validation as a part of parsing. While the second class includes JAXP Validation API, which could be used in those applications that need the process of validation and parsing to be decoupled. The experiment discussed in this paper uses the API of the second class, as the technique is proposed to support systems that separate validation from parsing. A number of scenarios might involve separating these two processes, for example: if the Schema was available from an external source, if the schema language was not supported by the available parser or if the application needed to validate multiple XML documents against the same schema definition. The JAXP Validation API was used to construct a separate validation program to support validating XML documents against external Schema whenever it is required.

References


Numerical Simulation of Ground Anchor-Soil Nail Retaining Systems for Academic-Learning Purposes

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Abstract

The ever-expanding urban architecture in developing areas requires more land space for construction purposes to be available. For this, utilizing the sub-surface areas through excavations in populous cities is now on the increasing trend. Two major concerns in such excavation projects are excavation-wall stability and the induced ground settlements which can be countered by a soil nailing-ground anchor system. In this regard, influential factors such as nail length and nail inclination angles can affect the overall performance of stabilized ground. Therefore, the focus of the present study is on how the aforementioned influence excavation-induced ground deformations. The numerical simulation is conducted using the software Plaxis 2D. The established numerical models help to explain how changes in the nails’ inclination angles and anchor lengths can change the observed behavior of the walls; from which helpful tips for practicing engineers are drawn accordingly. Such results could also be utilized for classroom presentations to aid students’ understanding of geotechnical engineering concepts.

Keywords: Numerical Simulation, Soil Nailing, Hardening Soil Model (HS), Fontainebleau Sand, Plaxis 2D.

1. Introduction

The construction procedure in large metropolitan areas and developing cities mandate more use of the land available. Since large structures such as skyscrapers and tall buildings would usually have foundations buried deep under the ground, the safe excavation of the surface soil is a significant part of these projects. This is mainly due to the lateral ground deformations and the settlements that occur as the outcome of these practices (Dong et al., 2014; Hsiung, 2009; Hwang et al., 2007; Konda et al., 2008; Liu et al., 2005). Therefore, the implementation of proper engineering standards and professional practice through appropriate measures to counter unfavorable outcome in the excavation procedure is a must. For this, different retaining systems such as pile walls, sheet pile walls, and soil nail walls are developed to control ground deformations in practice. Since the choice of the most suitable retaining system is project-specific, only apt engineering judgement would determine the design at last. However, economic considerations in every project would also play an important role in the decision-making processes. Eventually, engineers opt for a design that meets both safety and economy criteria. One of the most economically-viable options within the soil-retention techniques is soil nailing (Watkins and Powell, 1992). Soil nails are passive reinforcing elements that are inserted into the soil or soft rock and then grouted for further bonding with the surrounding domain (FHWA, 2015). The soil nails are sometimes combined with post-tensioned members to boost the overall performance of the retention system. These prestressed members are...
referred to as the tie-backs or ground anchors which comprise an active retaining system. The main purpose of such systems is to transfer tensile loads from the nails to the ground (FHWA, 1999). Since this technique eliminates the need to shore up the excavated wall, the work area will be clear of obstructions such as struts or any transverse elements. As a result, faster completion of the project in comparison to other techniques is quite feasible.

The conventional tools available for the design of soil nailed walls with anchor systems are based on the limit equilibrium method that can only capture the failure state of the structure (Barret et al., 2013). In this regard, some work has also been conducted to account for the interaction between the structural elements and the surrounding soil (e.g., Feijo and Erhlich, 2003; Pradhan et al., 2006). One group of techniques employed for the analyses concerning excavation-induced ground deformations include the empirical or semi-empirical methods (Clough and O'Rourke, 1990; Wong et al., 1997; Ng, 1998; Long, 2001; Moormann, 2004; Zhang et al., 2018). These methods arise from the fact that the purely mathematical approaches for practical applications in such problems are too complicated. Another alternative for a deep excavation problem is the numerical analysis method that enables the behavioral assessment of both structural elements and geotechnical domain under different loading, groundwater, and construction sequence conditions in projects (Hsieh and Ou, 1998; Yang and Drumm, 2000; Zhou et al., 2009; Khoiri and Ou, 2013; Likitlersuang, 2013; Garg et al., 2014; Nguyen and Treyssede, 2015; Shi et al., 2015; Orazalin et al., 2015; Hsiung et al., 2018).

Since the construction of soil-nailed structures are performed in stages, it can be deduced that the numerical tools available to tackle such problems are of profound importance for cost-effective analyses in projects. Consequently, engineers usually resort to numerical simulation studies that facilitate modeling investigations. For the current study, the developed numerical model was first validated by comparing its results with those of Wang et al. (2016) study. Following this, the 2D model of a vertical excavation wall was established and the variations of anchor length and drilling angle in the wall were modelled. The output results were then scrutinized and discussions on the effects of these factors on the overall system response were conducted. The results obtained in this work confirm the recommendations made in the available literature which are derived from monitoring observations and experimental or numerical investigations.

2. Numerical Simulation Procedure

2.1 Numerical Model
The software employed for the numerical simulation analyses is PLAXIS 2D V21.01 (2021) which is based on the finite element method for solving the partial differential equations in the study domain. Plane strain analysis is adopted for the numerical modeling of the excavation procedure. The sides of the soil domain are restrained in the horizontal direction only while pin supports are used for the bottom boundary. 15-node triangular elements are used in the meshing of the domain and no groundwater effects are considered for the depth of excavation.

2.2 Model Geometry
The geometric properties of the excavation model are chosen so that model geometry did not affect the accuracy of the results (Briaud and Lim, 1997). The height of the excavated wall is 10 m and the wall is nailed to the soil with 6-m nails. The first two nails are inserted into the ground after two 0.5-m excavation intervals. Other nails are then inserted at depth intervals of 1 m. All of the nails have a diameter of 36 mm and the ones located below the depth of 3 m in the wall are anchored to the ground using prestressed embedded beam elements. Figure 1 shows a schematic diagram of a soil retention system consisting of shotcrete wall, soil nails, and ground anchors. The specific geometry used in the current investigation is also depicted in the figure.

All the anchored nails are subjected to a prestressing force of 250 kN/m and the diameter of grouted anchorage in all cases is chosen as 30 cm. Three anchor lengths of 1, 2, and 3 m and four cases of drilling angles of 0, 5, 10, and 15 degrees are considered for the nailing system in this study to simulate real-world practices. A spacing of 1 m is considered for all the nails in the model which resulted in a square pattern for the anchored nails. Except for the two uppermost nails that are inserted at 0.5-m depth intervals, all other nails are placed at 1-m intervals throughout the depth of excavation. The first nail is anchored at the depth of 3 m in accordance with FHWA (1999) recommendation on the depth for the soil in front of the top anchor's bond zone. This is to prevent ground failure due to pullout forces. Additionally, the shotcrete wall thickness is 20 cm. Table 1 summarizes the geometrical and material properties of structural elements used in the numerical model.
Figure 1. Schematic Representation of a Soil-Nailed Wall in The Current Study Comprised of a Shotcrete Wall, Nails, and Anchors.

Table 1. Material Properties of The Structural Components of The Soil-Nailed Wall in This Study.

<table>
<thead>
<tr>
<th>Structural member</th>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shotcrete wall</td>
<td>Material behavior: Elastic</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Young's modulus (E)</td>
<td>GPa</td>
<td>21</td>
</tr>
<tr>
<td></td>
<td>Thickness (t)</td>
<td>cm</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>Poisson's ratio (ν)</td>
<td>-</td>
<td>0.2</td>
</tr>
<tr>
<td>Nails</td>
<td>Material behavior: Elastic</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Young's modulus (E)</td>
<td>GPa</td>
<td>210</td>
</tr>
<tr>
<td></td>
<td>Diameter (d)</td>
<td>mm</td>
<td>36</td>
</tr>
<tr>
<td></td>
<td>Drilling angle</td>
<td>°</td>
<td>0, 5, 10, 15</td>
</tr>
<tr>
<td></td>
<td>Spacing (L)</td>
<td>m</td>
<td>1</td>
</tr>
<tr>
<td>Anchors</td>
<td>Material behavior: Elastic</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Young's modulus (E)</td>
<td>GPa</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>Unit weight (γ)</td>
<td>kN/m$^3$</td>
<td>24</td>
</tr>
<tr>
<td></td>
<td>Diameter (d)</td>
<td>cm</td>
<td>30</td>
</tr>
<tr>
<td></td>
<td>Length</td>
<td>m</td>
<td>1, 2, 3</td>
</tr>
<tr>
<td></td>
<td>Spacing (L)</td>
<td>m</td>
<td>1</td>
</tr>
<tr>
<td></td>
<td>Axial skin resistance</td>
<td>kN/m</td>
<td>250</td>
</tr>
</tbody>
</table>

3 Soil Model and Soil Properties

The constitutive relationship in modeling soil behavior is of profound importance for obtaining accurate results. While many such models are available, the choice of the most suitable behavioral law is largely dependent on the problem studied. For the current investigation, the Hardening Soil (HS) model is used to characterize soil stress-strain relationship. The distinguishing feature of this model is that it is capable of modeling both soft and stiff soil behavior (Schanz, 1998). Since the plastic deformations in soil start from the early stages of loading, a hardening rule must be applied after the initial yielding in soil in order to better capture deformations (Plaxis, 2021). This rule ensures a stress-dependent stiffness for soil which is imposed through the input parameter (m) in a power law. The value of the power parameter in this study is assumed as 0.5 based on the earlier investigation by which this study is validated. It is worth mentioning that this parameter usually takes values between 0.5 and 1 with 0.5 as being suitable for sand. The HS model is superior to the hyperbolic model by Duncan and Chang (1970) as it uses theory of plasticity and a cap for the volumetric component of the yield surface for drained soil during triaxial tests. Additionally, the rule accommodates soil dilatancy as a factor that
affects its behavior. The input parameters for this model include the plastic straining due to primary deviatoric ($E_{50^{\text{ref}}}$) and compression ($E_{oed^{\text{ref}}}$) loadings, elastic unloading/reloading stiffness ($E_{ur^{\text{ref}}}$) and Poisson’s ratio ($\nu_{ur}$), cohesion ($c$), friction angle ($\phi$), and dilation angle ($\psi$).

The soil type used in the current study is Fontainebleau sand which is a well-sorted clean sand with particle diameters ranging from 0.063 mm to 0.25 mm (Latini and Zania, 2017). The HS parameters for the soil are chosen as those used in Sheil and McCabe (2016). Table 2 presents the Fontainebleau sand data employed in the current study.

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Parameter</th>
<th>Unit</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fontainebleau sand</td>
<td>Unsaturated unit weight ($\gamma_{\text{unsat}}$)</td>
<td>kN/m$^3$</td>
<td>16.5</td>
</tr>
<tr>
<td></td>
<td>Saturated unit weight ($\gamma_{\text{sat}}$)</td>
<td>kN/m$^3$</td>
<td>18.5</td>
</tr>
<tr>
<td></td>
<td>Minimum void ratio ($e_{\text{min}}$)</td>
<td>-</td>
<td>0.54</td>
</tr>
<tr>
<td></td>
<td>Maximum void ratio ($e_{\text{max}}$)</td>
<td>-</td>
<td>0.865</td>
</tr>
<tr>
<td></td>
<td>Deviatoric loading stiffness ($E_{50^{\text{ref}}}$)</td>
<td>kN/m$^2$</td>
<td>18e3</td>
</tr>
<tr>
<td></td>
<td>Compression loading stiffness ($E_{oed^{\text{ref}}}$)</td>
<td>kN/m$^2$</td>
<td>18e3</td>
</tr>
<tr>
<td></td>
<td>Unloading/reloading stiffness ($E_{ur^{\text{ref}}}$)</td>
<td>kN/m$^2$</td>
<td>45e3</td>
</tr>
<tr>
<td></td>
<td>Power (m)</td>
<td>-</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Cohesion ($c$)</td>
<td>kN/m$^2$</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Friction angle ($\phi$)</td>
<td>-</td>
<td>38</td>
</tr>
<tr>
<td></td>
<td>Dilation angle ($\psi$)</td>
<td>-</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Poission’s ratio ($\nu_{ur}$)</td>
<td>-</td>
<td>0.3</td>
</tr>
</tbody>
</table>

The critical depth of excavation for unsupported ground can be calculated from the following relationship (Das and Sobhan, 2014):

$$h_c = \frac{2c}{\gamma K_a}$$

in which $h_c$ and $K_a$ stand for the critical depth and Rankine’s active earth pressure coefficient, respectively. Considering the aforementioned in combination with a safety factor of 3 against ground failure, the unsupported depth for the excavated ground in the current study was obtained as 1.5 m. However, the first two layers of the excavated soil were limited to 0.5 m for limiting the ground deformation at the beginning stages of excavation.

4 Model Validation

The established numerical model was validated using the data obtained from CLOUTERRE project that was a national soil nailing research program in France throughout the years 1986-1990 (Plumelle and Schlosser, 1990). The soil type on site consisted of Fontainebleau sand with a uniform gradation. For the current study, a case study within the aforementioned project was chosen for the purpose of validating the numerical model. The wall in the chosen study had a height of 7 m with nails inserted at every 1 m and 1.15 m distances in the vertical and horizontal directions, respectively. However, the first nail was placed into the ground after a 0.5 m-thick soil layer was removed. The arrangement of the nails and their properties were selected so that no possibility of nail pullout existed. The nails had three different lengths of 6, 7.5, and 8 m with 16, 30, and 40 mm diameters. All the nails were inserted into the ground with an inclination angle of 10˚ with respect to the horizontal. The facing wall had a thickness of 8 cm and was constructed using shotcrete technology. Figure 2(a) shows details concerning model geometry and nailing configuration in one case study of the CLOUTERRE project. The results obtained from the numerical simulation study were compared with the measurement data from the project and a generally good agreement between the results was observed (Figure 2(b)). The software output plot illustrating variations in ground deformation within the domain is presented in Figure 2(c).
5 Results and Discussion

The numerical simulation analyses were conducted for the 10 m-wall using different drilling angles for the nails in combination with three different anchor lengths. The nail inclination angles in the study were 0, 5, 10, and 15 degrees from the horizontal direction which were considered for cases with 1, 2, and 3 m-long anchors. No surcharge loading was assumed in the computations and the ground water effects were ignored. Figure 3 illustrates the deformed facing wall and ground surface after running the model for the case of 1 m anchor length with an angle of 10 degrees with the horizontal.
The horizontal deformations of the wall and the accompanying ground settlements are significant factors in assessing the success of a completed project. Excess wall deformations are not only aesthetically unfavorable, but they could also pose serious hazards in populous urban areas. Furthermore, it is necessary for the vertical deformations in the immediate vicinity of the excavated ground to be limited so that no interruption in the services provided by infrastructure such as roads, buildings, and pipelines would materialize.

6 Drilling Angle and Anchor Length Variations
The effects of anchor length and drilling angle variations on the horizontal wall deformations and ground settlements were considered in this section. Figure 4 shows wall deformation graphs for different anchor lengths and nailing inclinations. The wall deformation magnitudes were observed to have a decreasing trend from stem to toe. Additionally, the deformations decreased with anchor length in all cases. These trends were found to be true for all cases regardless of the drilling angle of nails. Further investigation into the results revealed that the inclination angle of the nailing system is a factor in the amount of the deformations endured by the shotcrete wall (Figure 5). The wall deformations for the nailing system with no inclination with respect to the horizontal were the least and the values increased with nailing angle. Therefore, the most efficient drilling orientation for the nails is perpendicular to the wall (horizontal direction). However, practical limitations for drilling equipment and the favorable influence of gravity for grouting in the holes has rendered the nailing operation to be usually inclined at angles of 10 or 15 degrees with the horizontal.

Figure 6 shows the ground settlement profiles for different cases of anchor lengths within one drilling angle for each graph. It is noted that the effect of anchor length in reducing vertical deformations is significant for the first 15 m of distance from the facing wall. The longer anchors will render smaller ground settlements due to the excavation operation. However, it can be observed that the difference in ground settlements for all anchoring systems at distances larger than 15 m from the shotcrete wall is negligible. Similar to the discussion on the wall deformations, the smallest drilling angle lead to the most favorable outcome with the smallest settlements (Figure 7).
Figure 4. Wall Deformation Graphs for Different Anchor Lengths and Nailing Inclinations of (a) 0 degrees, (b) 5 degrees, (c) 10 degrees, and (d) 15 degrees.

Figure 5. Comparison of The Wall Deformation Profiles for Different Drilling Angles of Cases With (a) 1 m, (b) 2 m, and (c) 3 m Anchor Lengths.
Figure 6. Ground Settlement Graphs for Different Anchor Lengths and Nailing Inclinations of (a) 0 degrees, (b) 5 degrees, (c) 10 degrees, and (d) 15 degrees.

Figure 7. Comparison of The Ground Settlement Profiles for Different Drilling Angles of Cases with (a) 1 m, (b) 2 m, and (c) 3 m Anchor Lengths.

7 Conclusions
The parametric studies concerning geotechnical infrastructure have long been assisting engineers to improve their understanding of the unknown phenomena. While experimental investigations will comprise a substantial portion of research contributing to the field, the occasional costly and unwieldy apparatuses for such evaluations are significant...
drawbacks. Consequently, numerical modeling methods can be used as a substitute for the representation and assessment of problems with less financial burden. In the current study, the FHWA (1999) design recommendations for the design of anchored walls were evaluated through a number of parametric studies for an anchored wall. The results were first validated through earlier experimental investigations and were then extended to other scenarios. The observations can eventually be summarized as follows:

1) The horizontal displacements of the wall decreased from the wall stem to its toe. While the lateral wall displacements decreased with anchor length, the inclination angle of drilling could adversely affect the nails’ performance against lateral loading. Ideally, the best practice is for the nails to be placed at zero angle with the horizontal. However, limitations concerning drilling operations would usually lead to inclined nails with the drilling angles of about 10 degrees.

2) Ground settlements improved with the anchor length as less lateral deformation in the walls occurred. This was specifically noticeable in the vicinity of the wall stem. Nevertheless, the induced settlements in all anchor-length scenarios tended to converge to same values with distance from the stem.

3) The settlements far from the wall stem were found to follow a similar pattern for different nailing inclinations; same settlement values for all inclination degrees. The ground vertical deformations were also observed to be directly proportional to the drilling angles.

4) Although the results of the current study are focused on the deformation patterns of the wall and ground, other graphical presentations for different types of stress, strain, and constitutive relationships could easily be derived from every model built in the software. Additionally, a variety of influential factors such as soil strength parameters and geometrical aspects of the models could be scrutinized for further investigations. These evaluations would significantly reduce the costs associated with physical modeling and can expedite analyses with regard to any modifications in design and planning.

References


Impact of Carbon Fibre on Mechanical Characteristics of Clayey Soil Under Several Normal Stress

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Abstract
Geotechnical engineering requires the use of ecologically acceptable, long-lasting, and effective solutions to fortify clayey soil. The mechanical behavior of clayey soil strengthened with carbon fibres (CFs) was studied in this work. Soil specimens were subjected to uniaxial compression strength tests at their optimal moisture content (OMC). The impacts of CFs length and percentage on the strengthened soil specimens' shear resistance, and stress-strain curve behavior were investigated. The effect of CFs on specimen cohesiveness and angles of internal friction was also investigated. The results showed that adding CFs to clayey soil can increase its shear resistance and cohesiveness greatly. Because the fibres can be spread easily in soil samples and had a suitable length that can generate an interlaced network among soil grains that restricted soil movement once exposed to external stresses, it is presumed that utilizing three percent of CFs weight content had six millimeters length could indeed give the highest impact on resistance development among all the specimens.

Keywords: Clayey Soil, Carbon Fibres, Uniaxial Compression Strength, Shear Resistance, Mechanism of Failure, Angles of Internal Friction.

1. Introduction
During the building of foundations, road pavements, and slopes on softly clayey soil, many challenges such as excessive or uneven settling and inadequacy of load capacity might arise. Soil strengthening is a geotechnical engineering technique used to enhance the performance of dumpsite covers, heal shallow slope failures, and reinforce roadbeds, and similar. Cement, asphalt, lime, chemicals, and recycled materials have been once routinely used to enhance the mechanical behavior of softly soil. These admixtures, on the other hand, have been deemed to be both ecologically beneficial and long-lasting. (Terashi, 1980) investigated the behavior of cement-treated soil pile, and their findings revealed that the cement's reinforcing effectiveness degraded with time. (Quang and Chai, 2015) conducted permeability and consolidation tests on clayey samples treated with cement and lime. The findings revealed that adding a particular quantity of cement to the soil could increase the production of cementation products that plug the interaggregate gaps, lowering soil porosity. Low porosity, on the other hand, may delay the release of excess pore pressure held in the soil, resulting in local shear failure and jeopardizing the structure's integrity, particularly under cyclic stress circumstances. As a result, geotechnical engineers must devise new and more effective ways to fortify clayey soil. Many investigations on fibre-strengthened soil have been conducted (Consoli et al, 2013; Li et al, 2018; Liu et al, 2020) because the interface friction among fibre and
soil could enhance the mechanical characteristics of soil. (Mirzababaei et al, 2020) used a sophisticated X-Ray CT facility to explore the process of fibre re-orientation, locative, deformation, and tortuosity in a randomly fibre-strengthened clayey soil at diverse phases of stress. Their findings revealed a significant amount of anisotropic fibre arrangement (both in spatial position and angles in the XZ&XY directions), which is amplified by stress. Fibre parameters such as sort, incorporation percentage, length, length/diameter ratio, young's modulus, and direction are major influencing elements for soil strengthening (Hejazi et al, 2012; Amir-Faryar and Aggour, 2012; Yoo et al, 2017; Boz and Sezer, 2018). Carbon fibres (CFs) has a higher tensile resistance and young's modulus than traditional fibres like polypropylene (Pp). It also has high durability and a moderate biodegradation rate, making it ideal for soil strengthening. Carbon fibres have recently gained increased interest in civil engineering due to its extraordinary resistance, relatively wide length/diameter ratio and very good natural resistance to degradation. Nevertheless, the majority of research to date have mostly focused on the use of CFs in cement or concrete elements. Only a few researches have been published on the use of CFs for soil enhancement. Previous research revealed that randomly dispersed short CFs could efficiently strengthen non-cohesive soil (Cui et al, 2018). Finer soil grains, according to (Ranjan et al, 1996), could establish stronger interfacial adhesion with fibres due to a lower likelihood of slip failure than coarser soil grains. As a result, clayey soil expected to be able to form strong links with carbon fibres. Furthermore, because carbon fibres (CFs) have a smaller diameter (seven micron) than polypropylene fibres (PpFs), the length/diameter ratio of CFs is substantially greater. CFs are selected to strengthen soil since it has been claimed that the value of strengthened specimens rises gradually with increasing fibre length/diameter ratio for a particular fibre volume fraction (Yoo et al, 2017; ASTM Committee D2487-17 on Soil and Rock, 2017). Although PpFs is often less expensive than CFs, CFs is thought to be more effective since a tiny amount of CFs can result in significant soil improvement (Cui et al, 2018). As a result, the overall expense is not large when compared to the lower dose required for soil stabilization. Additionally, as fabrication technology improves, the cost of CFs is likely to decrease.

The purpose of this work is to see how CFs affects the mechanical characteristics of clayey soil. The impact of fibre lengths (three, six, and ten millimeters) and volume fractions (one, two, and three percentage by weight as a replacement of soil) on the mechanical behavior of soil samples was investigated using a set of direct-shear tests and uniaxial-compression tests. The mechanism of failure, ultimate resistance, and stress-strain curve behavior of the CFs strengthened soil samples were all thoroughly investigated and discussed.

2. Experimental Work
2.1 Materials and Method
The essential objective of this work is to assess the strengthening impacts of CFs of several lengths and volume fractions on mechanical features of clayey soil. Several experimental tests have been conducted which could be categorized into three categories:
• first category: water content & density relation test, direct-shear test under one hundred, two hundred, three hundred, and four hundred kilopascal standard stress and uniaxial compression test have been implemented on plane soil samples as reference samples.
• Second category: water content & density relation test, direct-shear test has been implemented on strengthened soil samples with several volume fractions of CFs (one, two, and three percent) and lengths (three, six, and ten millimeters) under one hundred, two hundred, three hundred, and four hundred kilopascal standard stress.
• Third category: uniaxial compression test implemented on strengthened soil samples with different volume fractions of CFs (one, two, and three percent). In this category, the utilized fiber length were only six millimeters.

2.1.1 Carbon Fibres (CFs)
China-made and locally available synthetic carbon fibres have been utilized as a strengthening material in the current experimental study (Figure 1). It has a diameter of seven micron and three different lengths (three, six, and ten millimeters). The tensile resistance is 4900 MPa, while its elongation, Young's modulus, and density are 2.1%, 230000 MPa, and 1.8 g/cm³, consecutively. According to the above excellent features, a small proportion of CFs was expected to efficiently improve the characteristic of soil.
2.1.2 Clayey Soil Samples
The soil employed in this work taken from Raparin excavation site, in the Sulaymaniyah governorate of the Kurdistan region of Iraq. The soil specimens have been collected from a depth of two meter below the level of natural ground and wrapped in plastic bags to prevent water evaporation while transporting to the laboratory for examining. Table 1 demonstrates the main features of the collected soil samples and the corresponding specifications that were adopted in the evaluation of the features. According to the unified soil classification system (ASTM, D2487, 2017) and the distribution curve of grain size (ASTM, D422, 2007), which is demonstrated in Figure 2, the soil could be classified as clayey soil. Figure 3 demonstrates the relation between water content and density, and it could be seen from the relation that the OWC equal to 27% with the corresponding MDD of 1510 kg/m³.

Table 1. Physical Features of Collected Plain Soil Samples.

<table>
<thead>
<tr>
<th>Features</th>
<th>Values</th>
<th>Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>MDD</td>
<td>1510 kg/m³</td>
<td>ASTM D 698</td>
</tr>
<tr>
<td>OMC</td>
<td>27%</td>
<td>ASTM D 698</td>
</tr>
<tr>
<td>Gs</td>
<td>2.6</td>
<td>ASTM D 854</td>
</tr>
<tr>
<td>Atterberg limits</td>
<td></td>
<td></td>
</tr>
<tr>
<td>LL</td>
<td>63.48%</td>
<td>ASTM D 4318</td>
</tr>
<tr>
<td>PL</td>
<td>29.13%</td>
<td></td>
</tr>
<tr>
<td>PI</td>
<td>34.35%</td>
<td></td>
</tr>
</tbody>
</table>

Figure 1. Scale of Utilized Carbon Fibers.

Figure 2. Grain size distribution of utilized soil.
2.1.3 Preparation of Samples
The samples could be taken in their natural moisture content (MC). However, the natural MC is typically low, which might lead to low soil plasticity, which prevents the homogeneous dispersion of fibres in the soil. The following procedure was used to solve this problem. The CFs have been blended with dry soil in a pan after being disseminated in deionized water (DI water). Considerable care has been given to assure that the CFs have been evenly dispersed throughout the mixture of soil. After that, the combination has been dried for twenty-four hours at one hundred and five degrees Celsius in an oven. Once done, the combination was crushed into powder and then whiskered with DI water (decided by the ideal moisture content) and maintained in a closed plastic bag for twenty-four hours to ensure that the moisture has been evenly distributed throughout the samples. Subsequently, the direct shear and uniaxial compression tests have been conducted according to ASTM D3080/D3080M and ASTM D2166/D2166M, consecutively, at a loading rate of 0.8 millimeters per minute.

3 Result and Discussion
3.1 Impact of Carbon Fibres Length and Volume Fraction on Shear Resistance Parameters
In comparison to the reference sample (natural plain soil sample), Figure 4 demonstrates the association between shear resistance and normal stress for samples with varying contents of three millimeters of CFs (volume fractions evaluated as a percentage by weight). The shear resistance appears to grow as the CFs concentration increases but begins to drop when more than two percent of CFs are applied, as shown in the graph. The samples with three percent of CFs had the maximum shear resistance among the three millimeters CFs samples. When exposed to normal stresses of one hundred, two hundred, three hundred, and four hundred kilopascals, their ultimate shear resistances were 85, 125, 170, and 214 kilopascals, consecutively. The impact of CFs on the resistance parameters is seen in Figure 5. The cohesiveness of samples containing two percent of CFs reached 41.1 kilopascals, which is 105.5 percent greater than the reference sample. The effect of CFs on angles of internal friction, on the other hand, was not substantial, with just a twenty percent enhancement was observed compared with the reference sample. When the samples were strengthened with more than two percent of CFs, both the cohesiveness and the angles of internal friction declined significantly. In most cases, adding CFs of three millimeters length to the soil sample would not result in a substantial improvement in tensile resistance.

The shear resistance of samples improved with increasing CFs concentration when CFs of six millimeters length was utilized, as illustrated in Figure 6. The higher amount of CFs which has length of six millimeter in the sample, the better the reinforcing impact on shear resistance appears to be (which is similar behavior in the case for CFs which has length of three millimeter). The impact of six millimeters of CFs on shear resistance and resistance characteristics is often greater than that of three millimeters of CFs. The angles of internal friction of samples with three percent content of CFs which has length of six millimeter was 24.6, exhibiting a 25.5 percent growth over reference samples, as illustrated in Figure 7. Their cohesiveness also improved dramatically, from twenty kilopascal to ninety-six kilopascal (an increase of three hundred and eighty percent). Both Figure 5 and 6 show that when the CFs proportion is increased from two percent to three percent, the reinforcing effect is stronger than when the proportion is altered from one percent to two percent.
Figure 4. Association Between Shear Resistance and Normal Stress for Samples with Varying Contents of Three Millimeters of CFs.

Figure 5. Impact of Three Millimeters of CFs on Angles of Internal Friction & Cohesiveness.

Figure 6. Association Between Shear Resistance and Normal Stress for Samples with Varying Contents of Six Millimeters of CFs.
This is in contrast to what was seen with three millimeters of CFs, where the resistance tended to diminish as the CFs concentration increased to three percent.

The impact of ten millimeters of CFs on shear resistance is comparable to that of six millimeters of CFs. The shear resistance of samples rose as the CFs concentration increased, as illustrated in Figure 8. Figure 9 shows the influence of ten millimeters of CFs on angles of internal friction and cohesiveness. In the case of samples with three percent of CFs, the greatest cohesiveness measured was 64.9 kilopascal. Despite the lesser improvement as compared to samples with six millimeters of CFs of the same content, a significant increase in cohesiveness was obtained, with a 224.5 percent growth when compared to reference sample.

The length and amount of CFs would, in general, have a significant impact on the mechanical characteristics of soil samples. The sample with a three percent content of six millimeters CFs had the greatest shear resistance values of all the samples examined, followed by the sample with a three percent content of ten millimeters CFs, and finally the sample with a two percent content of three millimeters CFs. (Kumar et al., 2006) observed comparable findings when soil samples were fortified in different ways employing varied quantities (0–2 percent) of three millimeters, six millimeters, and twelve millimeters polyester fibres (PLFs). Because of the fibre-soil contact friction, CFs was shown to be helpful in increasing angles of internal friction and cohesiveness. CFs is thought to generate mutual friction between the soil and the dense network structure, improving soil cohesiveness dramatically. As a result of the improved angles of internal friction and internal cohesiveness, soil shear resistance may be increased even more. Short-FCs strengthening, on the other hand, could fill part of the voids and also provide interconnecting impacts when dispersed equally in soil. The soil stiffness could also be improved by using short-CFs. CFs, which can inhibit the growth of stress fractures and soil deformation, may be responsible for the enhanced stiffness.
3.2. Uniaxial Compression Resistance

The connections between Uniaxial Compression resistance and axial strain for soil samples strengthened with various amounts of six millimeters of CFs are shown in Figure 10. The addition of CFs enhanced not only the ultimate shear resistance but also the post-peak residual resistances, as shown by the stress–strain relation. Before the ultimate point, the CFs enhanced the stiffness of the samples. The reference sample and the sample of one percent of CFs of six millimeters length both showed strain hardening behaviour, as shown in Figure 10. The stress-strain graphs of samples containing two and three percent of CFs of six millimeters length, on the other hand, showed quick hardening up to the maximum and subsequently softening after the maximum. The strengthened sample exhibited ductility when the CFs concentration was raised from two percent to three percent. As shown in Table 2, the young's modulus of the sample with three percent of CFs of six millimeters length was 5.29 MPa, which is 108.3 percent greater than the reference sample. As demonstrated in Figure 11, the resistance of samples appeared to rise as the concentration of CFs of six millimeters length increased. Maximum shear resistances of samples which contained one, two, and three percent of CFs of six millimeters length have been found to be greater than the reference sample by about 29.0 percent, 80.8 percent, and 122.8 percent. (Ding et al, 2018) showed similar enhancement rates when soil samples have been supplemented with twelve percent from total weight of cement. It's worth mentioning that raising the amounts of CFs can shift the stress-strain correlations of samples from strain hardening to hardening-softening behaviour. It could be concluded that CFs can enhance the connection between sand grains and restrict movement when the soil is affected by external stresses in the same way that cement can.

Figure 10. Association Between Uniaxial Shear Resistance and Axial Strain for Samples with Varying Contents of Six Millimeters of CFs.
Table 2. The percent of increment of maximum resistance and young's modulus for samples with varying contents of six millimeters of CFs.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Maximum resistance (kPa)/ increment (%)</th>
<th>Young’s modulus (MPa)/ increment (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ref.</td>
<td>142.5/ 0</td>
<td>2.55/ 0</td>
</tr>
<tr>
<td>1% FCs (6 mm)</td>
<td>171.41/ 28.95</td>
<td>2.90/ 14.5</td>
</tr>
<tr>
<td>2% FCs (6 mm)</td>
<td>257.55/ 80.76</td>
<td>4.78/ 87.8</td>
</tr>
<tr>
<td>3% FCs (6 mm)</td>
<td>317.42/ 122.8</td>
<td>5.29/ 108.3</td>
</tr>
</tbody>
</table>

Figure 11. Impact of Six Millimeters of CFs Content on Uniaxial Maximum Shear Loading.

4 Conclusions
The impacts of CFs volume fraction and length on features of clayey soil under series tests including direct shear test, uniaxial compression test have been investigated. The following conclusions can be drawn from this research:
1) With increased fibre percentage, the MDD and OMC both fell significantly. Adding CFs to soil samples can considerably increase their resilience.
2) The amount of resistance improvement is proportional to the length of CFs. Short fibres, such as CFs of three millimeters length, could not provide a substantial increase in resistance because there was little contact surface to connect with the sand grains, and the short fibres were quickly pulled out under shear loading.
3) Increased fibre percentage enhanced the uniaxial compression resistance of soil sample substantially. The samples with the highest failure resistance were those strengthened with three percent of FCs of six millimeters length.
4) Soil samples which strengthened with three millimeters, six millimeters, and ten millimeters of CFs enhanced their cohesiveness by 105.5 percent, 380 percent, and 224.5 percent, consecutively.
5) The percentage of CFs has a big impact on the resistance improvement. It would be challenging to homogeneously scatter the fibres in the soil sample when a high CFs percentage has been introduced, leading in cluster formation. The interfacial link would be weakened as a result of fibre aggregation, lowering shear resistance.

Nevertheless, additional study is needed to fully comprehend the reinforcing process of CFs on clayey soil, as well as its mechanical characteristics under saturated circumstances. Other significant clayey soil qualities, including permeability, time-based deformation, durability, and stability under various stress regimes, should also be addressed.

Reference


Preliminary Estimation of the Basic Reproduction Number of SARS-CoV-2 in the Middle East

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Abstract
Up to April 9, 2020, 142490 cases have been confirmed as COVID-19 infection including 5705 associated deaths in the Middle East. Most of the countries, such as Qatar, Bahrain, Iraq, Kuwait, United Arab Emirates (UAE), Oman, Lebanon, and Saudi Arabia have imported COVID-19 cases from Iran. Using the available data from WHO webpage, up to 9 April 2020, we traced epidemic curves and estimated the basic reproduction number ($R_0$) of COVID-19 through the susceptible-infectious-recovered (SIR) model for the Middle East countries. Epidemic curves for Middle East countries and territory show similar trend as Iran, with a couple of weeks’ delay in time. In SIR model, $R_0$ ranged between 7.41 as in Turkey to lowest as 2.60 for Oman whereas basic reproduction number for Iran, Kuwait, Bahrain, Qatar, Saudi Arabia, United Arab Emirates (UAE), Oman, Jordan, Egypt, Lebanon, Syria, Israel, West Bank and Gaza Strip territory, and Cyprus were 4.13, 2.71, 3.39, 4.18, 4.45, 2.75, 2.60, 3.52, 3.35, 3.16, 4.99, 4.08, 2.89, and 4.05, respectively. This study indicates an important trend on an early outbreak of COVID-19 based on estimated $R_0$ for the Middle East countries, mean $R_0 = 3.76$ for COVID-19, with median $= 3.51$. and interquartile range (IQR) $= 1.16$. in the Middle East.

Keywords: Basic Reproduction Number ($R_0$), COVID-19, Mathematical Model, SIR, Middle East.

1. Introduction
The COVID-19 infection, caused by the novel coronavirus SARS-CoV-2, is a contagious disease that can be transmitted through droplets, aerosols, and contact (Morawska and Cao, 2020; van Doremalen et al., 2020; Xu et al., 2020). The symptoms of COVID-19 infection appear after approximately 5.2 days (incubation period) (Li et al., 2020), which ranges from most common (fever, sore throat, cough, and fatigue) to variable ones (loss of smell, sputum production, headache, haemoptysis, diarrhoea, dyspnoea, and lymphopenia) (Huang et al., 2020; Li et al., 2020). In severe cases, infected cases may develop pneumonia, bronchitis, severe acute respiratory distress syndrome (ARDS), multi-organ failure, and death. The first confirmed COVID-19 case was detected in the Chinese city of Wuhan in late December 31, 2019, since then, the virus has spread worldwide. The World Health Organization (WHO) declared COVID-19 outbreak as a pandemic on March 11th, 2020 (WHO, 2020a). The reported cases of COVID-19 have since risen exponentially worldwide reaching more than 200 countries (Lewnard and Lo, 2020). As of April 9th, 2020, the outbreak has affected around 1521252 people.
globally and 142490 confirmed cases in the Middle East. The number of cases reported only in Iran until April 9th, 2020 is 68192 which records the highest number of infected cases of coronavirus in the Middle East. Iran has quickly led to an infection chain that represents the second highest COVID-19 outbreak after Italy, the epicenter outbreak in Europe, with local cycles of transmission have occurred in more than eight countries after imported cases.

In the emergence of a new infectious disease outbreak, predicting the trend of the epidemic is of crucial importance to suggest effective control measures and estimate how contagious it is and how far it could spread (Viboud et al., 2018). Mathematical modelling plays a key role in understanding the transmission rates and prediction evaluation of infectious diseases and their control measures since the early 20th century (Hamer, 1906). Thus, it can provide insights into the epidemiological characteristic which help decision makers to implement restriction strategies and prepare the health system capacities in the course of pandemic (Lauro et al., 2020; Wenbao et al., 2020). Rapid research on the prediction and transmissibility of COVID-19 has been focused on Asian and European countries (Yuan, 2019), with less attention in Middle Eastern countries. Increasing number of COVID-19 cases in Middle East, urged us to estimate the prediction and transmissibility in the region to implement appropriate alternative control measures accordingly.

The basic reproductive number ($R_0$) represents an indication of the initial transmissibility of the virus, i.e., the average number of secondary infections generated by each infected person. If this number is equal to one or less, $R_0 ≤ 1$, it indicates that the number of secondary cases will decrease over time and eventually the outbreak will peter out. However, when $R_0 > 1$, the outbreak is expected as transmission to secondary cases increases that urges the implementation of control measures.

Initially, WHO estimated the basic reproduction number for COVID-19 between 1.4 and 2.5, as declared in the statement regarding the outbreak of SARS-CoV-2, dated 23rd January 2020. Additionally, several articles aimed to more precisely estimate the COVID-19 $R_0$. Recently, (Liu, Y. et al, 2020) compared 12 published articles from January 1, 2020 to February 7, 2020 and they reported that mean and median estimates for $R_0$ were 3.28 and 2.79, respectively. Furthermore, Korolev, I (Korolev, 2020), reported $R_0$ to be 2 to 3 times higher for the US and Western countries than for Asian countries. Rahman, B. et al (Rahman et al., 2020) also reviewed 50 published articles which estimated 103 $R_0$ of COVID-19 with hovering between 0.32 and 6.47 in different countries including Italy, Iran, South Korea, Singapore, Japan, Israel, Algeria, Brazil and China. These differences are not surprising as transmission of COVID-19 multifaceted involving several changeable factors for deriving $R_0$ estimates, such as methods for modelling, variables to be considered, conditions on the clinical parameters, and various estimation procedures.

Up to our knowledge, limited number of articles published on prediction and $R_0$ of COVID-19 in Middle East. A group of researchers at Shahrekord University of Medical Sciences (Ahmadi et al., 2020), estimated $R_0$ to be 4.7 at the beginning of the outbreak in Iran and now $R_0$ has fallen below 2. Moreover, with three different scenarios estimated the prediction of confirmed cases by 3rd April 2020 at 19500, 27000, and 48830, with growth models of von Bertalanffy, Gompertz, and the least squared error, respectively. Authors concluded that the most ideal scenario is the Gompertz model. Current data from the WHO suggest more than a week ahead from Go

2. Method

Considering the daily data of the pandemic on the number of laboratory-confirmed cases reported from WHO situation reports, we employ an epidemic model SIR, a simple and widely used deterministic, that describes the flow of individuals through three mutually exclusive stages of infection: Susceptible ($S$), Infected ($I$) and Removed ($R$).
\[
\begin{align*}
\frac{dS}{dt} &= -\frac{\beta SI}{N}, \\
\frac{dI}{dt} &= \frac{\beta SI}{N} - \alpha I, \\
\frac{dR}{dt} &= \alpha I, \\
N &= S + I + R,
\end{align*}
\]

(1)

where \(\beta\) is the parameter that controls the transition between \(S\) and \(I\) and \(\alpha\) which controls the transition between \(I\) and \(R\). Removed compartment is a combination of both recovered and dead cases. In the absence of vaccine, the whole population assume to be susceptible, \(S \approx N\), in this case infected population can be approximated to

\[
\frac{dI}{dt} = \frac{\beta SI}{N} - \alpha I = (\beta - \alpha)I.
\]

(2)

Works out as

\[
I(t) = I_0 e^{(\beta - \alpha)t},
\]

(3)

where \(I_0\) is the number of infectious individuals at the beginning of the outbreak. An epidemic occurs if the number of infected individuals increases, i.e \(\beta - \alpha > 0\) which is equivalent to \(\frac{\beta}{\alpha} > 1\). According to the definition of \(R_0\), the transmissibility of a virus is measured by the basic reproduction number, which measures the average number of new cases generated per typical infectious cases, can be estimated throughout system (1), we arrive at the following inequality,

\[
R_0 = \frac{\beta}{\alpha} > 1.
\]

(4)

It is obvious that the basic reproduction number \(R_0\) is dependent on both infection rate \(\beta\), and removal rate \(\alpha\), which is the inverse of infectious period. As reported in the WHO-China joint mission on COVID-19 (Report of the WHO-China Joint Mission on Coronavirus Disease COVID-19, (2019)), in our simulation, we assume susceptible individual with COVID-19 takes an average of 7 days to be infected and takes an average of 14 days from infected to removed, yields \(\alpha = \frac{1}{14}\). Next, we used estimation procedure (least-square fitting) like Method 2 in (Chowell et al., 2007), and compute the residual sum of squares for the reported cases, given by

\[
RSS(B) = \sum_{t=1}^{n} (RD(t) - FD(t, \beta))^2,
\]

(5)

where \(RD\) stands for real data, and \(FD\) is a fitted data, that is the value of data \(t\) on the fitted curve, which can be predicted by SIR model, \(n\) is the number of daily reported cases of COVID-19. We then estimate the infection rate \(\beta\) by minimizing the \(RSS\).

3. Results and Discussion

The COVID-19 pandemic spread rapidly, and crossed borders reaching almost every corner of world and took more than two hundred thousand lives. The current economic and social impact of this pandemic is just the tip of iceberg. Due to lack of population immunity and an effective vaccine, the spread of COVID-19 is still expanding exponentially in many countries. Therefore, evaluating the effectiveness of implemented controlling measures like traffic restriction, lockdown, and social distancing are critical to stop the spread of the COVID-19 and save lives of thousands of people. Here we report the \(R0\) of all Middle East countries based on SIR model (Figure 1 and Table 1). Interestingly, it was found that Middle East countries seem to follow Iran’s trend with just a few week delays in time.
Figure 1. Dot Chart Showing the Estimated $R_0$ in the Middle Eastern Countries and Territory.

Table 1. The Basic Reproduction Numbers and the Corresponding Key Parameters.

<table>
<thead>
<tr>
<th>Countries/Territory</th>
<th>Population</th>
<th>n</th>
<th>$\beta$</th>
<th>$R_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Iraq</td>
<td>40,222,000</td>
<td>46</td>
<td>0.25012</td>
<td>3.50168</td>
</tr>
<tr>
<td>Turkey</td>
<td>83,200,000</td>
<td>30</td>
<td>0.52979</td>
<td>7.41706</td>
</tr>
<tr>
<td>Iran</td>
<td>82,914,000</td>
<td>50</td>
<td>0.2952</td>
<td>4.1328</td>
</tr>
<tr>
<td>Kuwait</td>
<td>4,700,000</td>
<td>46</td>
<td>0.19422</td>
<td>2.71908</td>
</tr>
<tr>
<td>Bahrain</td>
<td>1,443,000</td>
<td>46</td>
<td>0.2425</td>
<td>3.395</td>
</tr>
<tr>
<td>Qatar</td>
<td>2,839,000</td>
<td>40</td>
<td>0.29878</td>
<td>4.18292</td>
</tr>
<tr>
<td>Saudi Arabia</td>
<td>34,140,000</td>
<td>39</td>
<td>0.31805</td>
<td>4.4527</td>
</tr>
<tr>
<td>UAE</td>
<td>9,680,000</td>
<td>50</td>
<td>0.19549</td>
<td>2.73686</td>
</tr>
<tr>
<td>Oman</td>
<td>5,107,000</td>
<td>45</td>
<td>0.18585</td>
<td>2.6019</td>
</tr>
<tr>
<td>Jordan</td>
<td>10,458,000</td>
<td>39</td>
<td>0.25188</td>
<td>3.52632</td>
</tr>
<tr>
<td>Egypt</td>
<td>100,388,000</td>
<td>50</td>
<td>0.23971</td>
<td>3.35594</td>
</tr>
<tr>
<td>Lebanon</td>
<td>6,825,000</td>
<td>48</td>
<td>0.20647</td>
<td>3.31607</td>
</tr>
<tr>
<td>Syria</td>
<td>10,070,000</td>
<td>18</td>
<td>0.28999</td>
<td>3.99686</td>
</tr>
<tr>
<td>Israel</td>
<td>8,972,000</td>
<td>49</td>
<td>0.20647</td>
<td>3.89058</td>
</tr>
<tr>
<td>West Bank &amp; Gaza Strip</td>
<td>5,101,000</td>
<td>36</td>
<td>0.20647</td>
<td>2.89058</td>
</tr>
<tr>
<td>Cyprus</td>
<td>1,207,000</td>
<td>31</td>
<td>0.28999</td>
<td>4.05986</td>
</tr>
</tbody>
</table>

The control and delay in spreading of COVID-19 infection strongly rely on the capabilities of a country’s health system. The health system of many developing countries in the Middle East is below the world’s standards (Ghsindex, 2020) that make these countries vulnerable to a rapid spread of COVID-19 with higher rate of fatality as a result of current pandemic. It is worthy to note that, current official cases in the region is far behind those in the US, Italy, and Spain.

The overall timeline of the outbreak has provided countries with a window of opportunity for the early implementation of preventive interventions. Thereby, the before math intervention introduced in some Gulf countries play a considerable role in slowing down the number of confirmed cases, or “flattening the curve” of the virus’ growth (Vatanka et al., 2020). A recent article illustrates that the government response and population demography in Iraq have a significant role in flattening the epidemiological curve (Jebri, 2020). We suggest all countries need to consider the Chinese and Italian
lessons and implement immediate and more powerful control measures in order to stop the spread and break the chain of transmission.

Regarding the used parameters by SIR model for the prediction, the $R_0$ values shown in Figure 1, is relatively consistent with several mathematical models that have been used so far. Estimated the COVID-19 $R_0$ varies from 1.69 to 6.47 (Muniz-Rodriguez et al., 2020). A review written by (Liu et al, 2020) compared 12 published articles from the 1st January to the 7th of February 2020 which estimated for the $R_0$ for COVID-19 a range of values between 1.5 and 6.68. In a recent article, (Jiang, S et al, 2020)(Wang et al., 2020), estimated $R_0$ to be 6.66, 5.03, 5.60, and 8.93 for the four main epicenters Wuhan, Korea, Italy and Iran respectively. (Sardar, et al, 2020), estimated $R_0$ in different Indian states (Maharashtra, Delhi, Tamil Nadu) and overall, India ranged between 1.46 to 8.44. By comparing the various investigation, these different basic reproduction number arise for many reasons: the virus is shed before symptoms begin, it is hard to identify the incubation period, the proportion of infected cases missed at the tracing and control procedures and the effectiveness of current strategies to prevent the spreading of the infection (Viceconte and Petrosillo, 2020).

Based on current model, the trend falls into three different curves which are exponential, sub-exponential, and polynomial (Figure 2) and number of cases in each country shown in (Figure 3). In Iran, Turkey, Saudi Arabia, and Israel, the confirmed cases rise exponentially; in Qatar, UAE, and Egypt increase sub-exponentially; in Iraq, Kuwait, Bahrain, Oman, Jordan, Lebanon, Syria, West Bank and Gaza Strip, and Cyprus grow polynomially. These countries have exponential curves recorded the highest $R_0$ ranging from 4.08 to 7.41, while countries and territory with polynomial curves reported the lowest ranging between 2.60 to 4.05.

Figure 2. Cumulative of Confirmed Cases for Middle East Countries and Territory, with Estimated Delay in Time from Iran’s Situation, as of 9th April 2020. Black dots are for countries with less than 1 week delay in time from Iran; red is for countries with 1–2 weeks delay in time; blue is for countries and territory with 2–3 weeks delay in time; and green is for countries with more than 3. The Iranian and Turkish data curves are cut at 2100 cases in order to compare delay in time with other countries. (WHO, 2020)
4. Conclusions

Basic reproduction number of COVID-19 is crucial parameter during a pandemic which is used to estimate the risk of COVID-19 outbreak and evaluate the effectiveness of implemented measures. For the first time, we report estimates of the basic reproduction number $R_0$ of COVID-19 outbreak in the Middle East countries and territory together using SIR model by fitting the model to official reported data from WHO. It is also observed the Middle East countries epidemic curves is just a couple of weeks behind Iran. According to the result Turkey has the highest $R_0$ and Oman has the lowest.

This study also found that the estimated mean $R_0$ for COVID-19 is around 3.76, with a median of 3.51 and IQR of 1.16. This mean $R_0$ is close to the two recent review articles by (Liu et al, 2020 and Alimohamadi, Y. et al, 2020) which estimated $R_0$ to be 3.38 and 3.28 respectively. Due to short onset time, current estimates of $R_0$ for COVID-19 in the Middle East might be biased. Moreover, the basic reproduction number is continuously modified during a pandemic by accurate assumptions introduced and becomes more reliable $R_0$ as more data and information come to light. The limitations of this study as it is applied to all other similar studies, due to limited number of tests in the whole population, the number of asymptomatic cases which may account about 25% of the population have been excluded and the reported confirmed cases are believed to be lower than the actual cases.

In summary, this study provides important findings on an early outbreak of COVID-19 in the Middle East. In theory, most the Middle East countries are in better positions than many other countries to react to the current outbreak. Nevertheless, health care system in some countries and territories of Middle East may face serious challenges as COVID-19 cases overwhelm hospitals, therefore, governments should work closely with the WHO to visit countries in need to provide recommendations, support control and prevention efforts. In the meantime, the Middle East countries and
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Mapping Geotechnical Soil Properties of Ranya City in Kurdistan Region of Iraq Using GIS

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Abstract
Geotechnical map is a vital guidance to visualize the behavior of soils. The objective of this paper is to present the geotechnical maps that can be used for preliminary investigation in Ranya city of northern Iraq. The study area is 13.02 km² with latitude and longitude of 36°15'14" N 44°52'59" E, respectively. A total number of 116 boreholes with the depth up to 5.0 m were utilized to create allowable bearing capacity, particle size, and Atterberg limit maps. Kriging interpolation tool in the ArcGIS software was used to analyze the soil properties data and to achieve the maps. The appraisal study area was divided into three layers 0.5-1.5, 1.5-3.0, and 3.0-5.0 m and the results show the average bearing capacity of 112.2, 168.5, and 244.2 kN/m² sequentially. Moreover, Particle size distribution’s results illustrate that gravel percentage increases in the deeper layers, while fines content decreases with no significant change of sand content. In addition, very high bearing capacity areas were mostly found in the southern and northern parts of the studied area. However, the eastern area represents the area with the minimum bearing capacity where it gradually increases toward the west. Furthermore, the liquid limit and plasticity index reduce from the north to south with an increase in depth of the layers from 3.0-5.0 m. The highest liquid limit value is observed in the depth of 1.5-3.0 m.

Keywords: Atterberg Limit Map, Bearing Capacity Map, Geotechnical Map, GIS, Particle Size Map.

1. Introduction
Rapid population growth demands more residential areas for settlement. In the past 15 years, the population of Ranya district has increased rapidly by 60 % (Annual population statistical report, 2021). This population rise increases the need for building more residential areas and engineering projects, for which site investigation of the soil is essential which is a difficult task that requires substantial effort, time and cost.

Using technology can save time and cost of the investigations if used correctly. With the developments in computation in the last two decades, GIS is often used to capture, store, manage, query, analyze, display and retrieve huge volumes of referenced geospatial data and associated attribute data collected from variety of sources (Manguri and Hamza, 2021; Mirzahossein et al., 2020; Tripathi et al., 2021). GIS provides data with actual geographical location. It has a significant role in geotechnical engineering, for instance; generating and visualizing data of inaccessible locations with the aid of interpolation techniques and representing the analyzed data by means of digital maps, graphs, and charts (Singh et al., 2015). In addition, GIS multifunctional features especially spatial and statistical operations can be used to perform analysis.
and visualize types of composites in hard and soft copy formats such as images, notices, borings, geotechnical properties of soils, and other site-specific data.

Many researchers tried to create GIS maps for a local area such as; (Jimenez et al., 2000) investigated hazards map prediction of Barcelona in Spain that revealed predominant period, amplification ratio and geology maps. In addition, (Valverde-Palacios et al., 2014) demonstrated a geotechnical map of foundation conditions for Granada in Spain based on properties of soils such as standard penetration value, bearing capacity, cohesion, internal friction angle, bulk density, particle size, groundwater table condition and minimum foundation depth. The problematic and expansive soil layer maps of the Toshka region in Egypt and Surfers Paradise in Australia were determined by (Labib and Nashed, 2013) and (Al-Ani et al., 2013) respectively. Other researchers investigated some physical and mechanical properties. For example, some of municipal districts of Tehran in Iran are presented by digital maps (Razmyar and Eslami, 2016, 2018). (El-Kady and ElMesmary, 2018) explored and formed a database for soil and rock types of 12 m below ground surface and interpolated the data using GIS to create a bearing capacity map of the Sakaka area in Saudi Arabia. (Kaur et al., 2019) calculated bearing capacity mapping of Srinagar, Jammu and Kashmir utilizing 39 boreholes.

(Kamal et al., 2016) and (Khalid et al., 2021) developed geotechnical map zones of Pakistan based on standard penetration values for Faisalabad and Islamabad. Also, (Arshid and Kamal, 2020) exposed soil classification, bearing capacity and AASHTO subgrade rating at different depths for Potohar plateau in Pakistan. (Celik et al., 2021) presented and investigated geological and geotechnical data for Nigde city in Turkey dividing the area to five regions. They plotted standard penetration values, bearing capacity, liquefaction zone, soil classification and plasticity index maps. (Cabalar et al., 2021) investigated Erzincan city near north Anatolian fault zone in the eastern Turkey, a series of geotechnical maps based on data from 92 boreholes were produced. The maps included water content, Atterberg limits, soil classification, standard penetration test, shear wave velocity, and primary wave velocity. Also, a relationship between the unconfined compression testing results and dynamic elastic modulus was achieved by neural network.

Some other researchers demonstrated the physical and mechanical properties of soil by producing digital maps in the middle and south of Iraq, for instance; (Kadhim et al., 2013) focused on distributed geotechnical properties of Basrah City through presenting bearing capacity, undrained shear strength, liquidity index and compression index maps at different depths. (Kadhim and Al-Abody, 2015) created bearing capacity maps to residential areas for every 1.5 m of depth up to the depth of 10 m of Al-Imam district in Babil, Iraq. Similarly, (Al-Maliki et al., 2018) showed the bearing capacity map of An-Najaf and Kufa cities at depths of 0-2 m with ranges of 5-20 Ton/m². Also, (Aldefae et al., 2020) created digital geotechnical parameters including bearing capacity, coefficient of consolidation, compression index, swell index, cohesion, angle of friction and groundwater level for Wasit province, Iraq, using 164 borehole logs distributed over 17000 km² studied area with the range of the depth is 0-10 m. (Ali and Shakir, 2021) studied soil characteristics in ThiQar Governorate, Iraq, based on 423 boreholes distributed over an area of 12900 km². They drew geotechnical maps comprised of water content, dry density, liquid limit, specific gravity, standard penetration test, groundwater and ultimate bearing capacity maps. In Salah Al-Deen, digital maps were produced by (Mohammed et al., 2020) representing the distribution of soil classification, bearing capacity, standard penetration test and chemical properties of the soil. Meanwhile, (Al-Mamoori et al., 2020) created unified soil classification system, and coarse and fine grained maps using 464 boreholes for An-Najaf city at depths of 0-26 m covering an area of approximately 105.1 km². Additionally, researchers have studied characteristics of soil properties in Sulaimani governorate in the north of Iraq. (Najmaldin et al., 2020; Rashed and Hussein, 2020). Physical and mechanical soil characteristics of Sulaimani city have been studied by using ArcGIS for characterization and modeling purpose based on the experimental and literature data (Ahmed et al., 2020).

In accordance with the literature data, geotechnical maps were created for different places in Iraq, which are vital for construction process. However, there is no such a geotechnical map which was studied for Ranya district, and it is considered as a research gap. Since soil investigation is costly and cannot be done for every small project, the engineers and designers need to assume some geotechnical parameters of the soil especially bearing capacity value. As a reasonable solution for the issue of lack of similar research in the study area, the authors of this study collected and used the available data from technical reports to create geotechnical maps for different soil properties of Ranya city. Similar to other studies found in the literature, this study has used GIS as a tool for producing the maps. However, the study tries to cover most of soil properties including the bearing capacity. Also, it tries to utilize more borehole data to represent the maps accurately.
2. Study Area
Ranya district is located in Sulaymaniyah governorate, Kurdistan Region, Iraq, with latitude and longitude of 36°15'14"N 44°52'59"E respectively. It is located in the north west of Sulaymaniyah city as shown in Figure 1. Its elevation is about 615 m with respect to the MSL. Ranya is approximately 104 km away from Sulaymaniyah city center (Bapeer et al., 2020).

The area of Ranya municipality is about 27.15 km² while the study area is only 13.02 km² which is part of most populated area of the city. Structurally, Ranya is located in the boundary between highly folded zone and imbricated zone (Jassim and Goff, 2006). Geologically, the district area is covered by alluvial sediments, which consist of gravel, sand, silt, and clay (Bapeer et al., 2020; Sissakian, 2000). The groundwater table ranges between (20-40) m below the ground surface (Al-Jiburi and Al-Basrawi, 2015; Al-Manmi, 2008). The city is surrounded by two mountains namely; Kewerash from the north, Hajila from the west, and Dokan lake from the south. Ranya climate belongs to Mediterranean system climate so, its winter is cold and wet while its summer is hot and dry (Khdir and Saeed, 2021).

Figure 2. Ranya City in The Iraqi Map (Study Area).

3. Methodology
3.1. Data Collection
Geotechnical investigation reports were collected from 26 sites providing data of 116 boreholes distributed over the study area. The tests were carried out by consultant engineering bureaus (Consultant Engineering Bureau of Sulaimani University; Seko Private Construction Lab; Sulaymaniyah General Construction Lab). The data were implemented for infrastructure projects of Ranya municipality boundary. The boreholes range up to 5 m. From each of the boreholes, disturbed and undisturbed samples were collected for performing tests to determine mechanical properties of soil according to the ASTM standards. The extracted data from the reports were Atterberg limits ASTM D4318 (ASTM, 2010), particle size analysis ASTM D422-63 (ASTM, 2007), unit weight ASTM D7263 (ASTM D7263, 2018), standard penetration test ASTM D1586 (ASTM D1586-11, 2011) and dynamic cone penetrometer ASTM D6951 (ASTM D
D6951, 2003). The study area was divided into three layers with depths ranging between 0.5-1.5, 1.5-3.0, and 3.0-5.0 m. For each layer, liquid limit (LL), plasticity index (PI), gravel, sand, fines content (silt and clay) ratios and bearing capacity are found and plotted. Each parameter indicates the number of data, ranges (maximum and minimum), average and standard deviation. These geostatistical data are summarized in Table 1. After that the data set was arranged, tabulated, digitized and georeferenced to analysis by ArcGIS 10.8 software to achieve digital maps.

Table 2. Statistical Parameters of Geotechnical Properties of The Study Area.

<table>
<thead>
<tr>
<th>Depth 0.5-1.5 m</th>
<th>Gravel %</th>
<th>Sand %</th>
<th>Fines %</th>
<th>LL %</th>
<th>PI %</th>
<th>Bearing Capacity kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of data</td>
<td>63.0</td>
<td>63.0</td>
<td>63.0</td>
<td>82.0</td>
<td>82.0</td>
<td>106.0</td>
</tr>
<tr>
<td>Max</td>
<td>82.4</td>
<td>46.0</td>
<td>99.0</td>
<td>59.9</td>
<td>30.2</td>
<td>207.0</td>
</tr>
<tr>
<td>Min</td>
<td>0.0</td>
<td>1.0</td>
<td>6.0</td>
<td>18.0</td>
<td>5.0</td>
<td>56.0</td>
</tr>
<tr>
<td>Mean</td>
<td>22.0</td>
<td>17.7</td>
<td>60.3</td>
<td>45.2</td>
<td>15.2</td>
<td>112.2</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>24.5</td>
<td>12.2</td>
<td>7.3</td>
<td>6.3</td>
<td>36.8</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth 1.5-3.0 m</th>
<th>Gravel %</th>
<th>Sand %</th>
<th>Fines %</th>
<th>LL %</th>
<th>PI %</th>
<th>Bearing Capacity kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of data</td>
<td>83.0</td>
<td>83.0</td>
<td>83.0</td>
<td>71.0</td>
<td>71.0</td>
<td>114.0</td>
</tr>
<tr>
<td>Max</td>
<td>80.0</td>
<td>45.0</td>
<td>96.0</td>
<td>59.2</td>
<td>30.5</td>
<td>321.0</td>
</tr>
<tr>
<td>Min</td>
<td>0.0</td>
<td>0.0</td>
<td>4.4</td>
<td>16.0</td>
<td>3.0</td>
<td>80.0</td>
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<tr>
<td>Mean</td>
<td>35.0</td>
<td>20.3</td>
<td>44.8</td>
<td>39.9</td>
<td>14.5</td>
<td>168.5</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>28.3</td>
<td>12.7</td>
<td>12.4</td>
<td>7.5</td>
<td>51.4</td>
<td></td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th>Depth 3.0-5.0 m</th>
<th>Gravel %</th>
<th>Sand %</th>
<th>Fines %</th>
<th>LL %</th>
<th>PI %</th>
<th>Bearing Capacity kN/m²</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. of data</td>
<td>82.0</td>
<td>82.0</td>
<td>82.0</td>
<td>59.0</td>
<td>59.0</td>
<td>108.0</td>
</tr>
<tr>
<td>Max</td>
<td>84.0</td>
<td>54.0</td>
<td>96.0</td>
<td>56.0</td>
<td>27.4</td>
<td>417.0</td>
</tr>
<tr>
<td>Min</td>
<td>0.0</td>
<td>0.0</td>
<td>2.3</td>
<td>18.0</td>
<td>3.0</td>
<td>116.0</td>
</tr>
<tr>
<td>Mean</td>
<td>41.7</td>
<td>17.3</td>
<td>40.9</td>
<td>32.7</td>
<td>10.3</td>
<td>244.2</td>
</tr>
<tr>
<td>Std. deviation</td>
<td>29.3</td>
<td>11.2</td>
<td>28.1</td>
<td>11.5</td>
<td>5.0</td>
<td>70.0</td>
</tr>
</tbody>
</table>

3.2. Interpreting Data for Bearing Capacity Calculation

In the most geotechnical reports, standard penetration test (SPT) is given, while dynamic cone penetration (DCP) is provided in some other reports. All values of SPT and DCP were converted to bearing capacity. According to (Meyerhof, 1956), the allowable bearing capacity for 25mm settlement can be obtained from SPT-N value using Equation (1):

\[
q_a = \frac{N}{0.08} \left( \frac{B+0.3}{B} \right)^2 \times w \gamma \times R_d
\]

Where, \(q_a\) = allowable bearing capacity kN/m², \(N\) = SPT blow number, \(w\) = water table correction factor = 0.5 + 0.5\(\frac{b}{B}\) ≤ 1, \(b\) = depth of water table below the base (m), \(B\) = base width of footing (m), for this research \(B\) is taken as 2 m, \(R_d\) = depth correction factor = 1 + 0.33\(\frac{D_f}{B}\) ≤ 1.33, \(D_f\) = foundation depth (m).

Some of the geotechnical reports utilized DCP instead of SPT for determining the bearing capacity of cohesive soil (Alshkane et al., 2020) proposed Equation (2) where Unconfined Compressive Strength (UCS) is derived from the DCP.

\[
UCS = 1033.6 \times DCP^{-0.968}
\]

Bearing capacity of the soils was calculated according to Equation (3) developed by (Terzaghi et al., 1996), for a foundation of undrained saturated soil when internal friction angles are zero.

\[
q_u = 5.7 \times c_u + \gamma \times D_f
\]

Where, \(q_u\) = ultimate bearing capacity (kN/m²), \(c_u\) = undrained shear strength (kN/m²), \(\gamma\) = unit weight of soil (kN/m³).

The allowable bearing capacity can be obtained by dividing the found ultimate bearing capacity by a factor of safety. The minimum factor of safety that can be used is 3.

3.3. Software and Management

The collected data were tabulated (sorted and categorized) in Microsoft Excel software which is recognized by the GIS environment. This includes the name of the studied site and the site’s features. The borehole locations are spatially determined using base maps (imagery with labels) in ArcMap 10.8.
For the purpose of data analysis and production of digital maps, the method of kriging spatial-analyst interpolation tool is used.

4. Results and Discussions

According to the variation of geotechnical properties of the soil at the area, thematic maps have been driven for three different depths of the boreholes; 0.5-1.5, 1.5-3.0, and 3.0-5.0 m. In each depth, a digital map is achieved for soil bearing capacity, gravel percent, sand percent, fines percent, liquid limit, and plasticity index. They are divided into six classes and presented in different colors.

4.1. Particle Size

Particle size is one of the important parameters in soil classification, particle characteristics significantly influence the soil strength (Koerner, 1970). Generally, the angle of shearing resistance increases as the median particle diameter and gravel content increment (Wang et al., 2013). Fines content that includes silt and clay percentage of soil has a tremendous influence on the value of the soil strength and its settlement. Dominant fine content has higher plasticity and compressibility, greater potential swelling and lower hydraulic conductivity (Mitchell et al., 2005).

Figure 2a, 2b, and 2c show the variation of the gravel across the study area in the depths of 0.5-1.5, 1.5-3.0 and 3.0-5.0 m respectively. Average gravel content in the first, second and third layers are 22%, 35% and 41.7% respectively. The lowest gravel range is 0.0-12.8% which is indicated in dark red color and it is found to be in the eastern part of the study area as shown in Figure 2a, while the gravel ratio increases gradually towards the western part that reaches the maximum range of 65.7-76.7% for the first layer. In the second layer, the dominant zone is the range of 42.2-61.1% that depicted in light green color, also some small spots were created that included the maximum range of 61.2-75.4% due to variation of the borehole values. However, the ranges of 32.6-42.1% and below cover the north-east and the north-west parts, the gravel ratio increases moderately as illustrated in Figure 2b. In the third layer, the gravel ranges of 38.7-49.5% and 49.6-60.4% covered most parts of the studied area from the south to the north, although other ratios from the central to the border sides gradually decreased as displayed in amber to the dark red colors in Figure 2c.
The average sand contents are 17.7%, 20.3% and 17.3% in all the three layers sequentially. Its ratio increased by 15% in the second layer compared to that of the first layer, while in the third layer the ratios are not significantly changed. Generally, the majority areas in all layers of sand percent covered by yellow and light brown colors with different patterns. Other ratios have been covered by a small area for each of depths, which are mostly available in the surrounding of the study area as in the Figure 3a, 3b, and 3c.
In the first layer, most of the study area can be considered as cohesive soil with the average fines content of 60.3% as demonstrated in dark blue, blue and green colors. It can be noticed from Figure 4a the fines content gradually increases toward the east of the study area, since the region is plain and agricultural land. In the second and third layers, fines content decreases compared to that of the first layer as the ratio decreases to 25.7% and 32.2% respectively. Predominant values of the second and third layers are represented in light brown and yellow colors zone as shown in Figure 4b and 4c.

To sum up, gravel dramatically increases with an increase in the depth of the layers, while fines content decreases. On the other hand, sand content was nearly the same.
4.2. Atterberg Limits
The Atterberg limits are used to classify fine grain soil and to investigate some other geotechnical characteristics of soil such as compression index, soil shrinkage rate and swelling potential (Zhou and Lu, 2021). According to the gathered data from different boreholes, the values of the liquid limit and plasticity index in the study area vary from a place to another as displayed in Figure 5 and 6. The highest liquid limit value of 49.9-57.4 % is observed in the depth of 1.5-3.0 m, which is hatched in dark blue color in Figure 5b. However, the lowest liquid limit value of 17.5-24.5 % is observed at the south-east of the study area at the same depth, which is hatched in dark brown color that covers a small area. The dark blue color in Figure 5a indicates a high liquid limit range of 49.3- 57.1 %. In Figure 5b the majority of the area in the depth of 1.5- 3.0 m is hatched in dark blue color in the middle, north and west of the study area which is high liquid limit area. The low value of liquid limit ranged from 20.0-27.8 %, it can be noticed in the depth of 3.0-5.0 m of the area compared to two other depths as displayed in Figure 5c.
Similar to the liquid limit values, the plasticity index (PI) percentage which is the difference between liquid limit and plastic limit varies from a place to another in the study area. The lowest percentage of PI of 3.9-7.3 % is noticed in depth of 1.5-3.0 m which is in the south part as indicated in Figure 6b and is hatched as light-yellow color. However, the highest percentage of PI of 26.0-29.1 % is observed at the same depth in the north part of the studied area. This range is hatched in dark brown color. The PI value is high in the north to the center of the area while it becomes lower in the south to the east in all depths. It is low in the depth of 3.0-5.0 m in the most of the studied area as shown in Figure 6c.
4.3. Bearing Capacity

Bearing capacity changes horizontally and vertically across the thematic maps. The bearing capacity increases with an increase in the depth of the layer. Figure 7a shows the bearing capacity at a depth of 0.5-1.5 m. The dark blue color represents the high bearing capacity areas which mostly covers the southern and northern areas of the study. The highest range of the bearing capacity of that layer is between 143.1-182.4 kN/m². The eastern area has the minimum bearing capacity ranging from 74.7 to 95.3 kN/m². This is shown in red color that gradually changes to the orange and yellow colors pointing to the increase of bearing capacity toward the west.
In the depth of 1.5-3.0 m, the abundant range of bearing capacity is 157.0-182.6 kN/m², which is represented by yellow color. The orange color zone in Figure 7b represents the lower range of bearing capacity between 131.3 to 156.9 kN/m² that is extended from east to the north west of the study area. Due to the variation of soil properties, three areas show the highest value of bearing capacity in that layer in light blue color with the range of 182.7-208.2 kN/m².

The third layer which is at a depth of 3.0-5.0 m is depicted in Figure 7c and mainly hatched by two bearing capacity zones. The first zone is orange with bearing capacity of 213.2-239.4 kN/m² which is located in the east, west, and south. The second zone is yellow with bearing capacity of 239.5-265.6 kN/m² that is located in the center of the study area. The bearing capacity gradually increases until it reaches the maximum range of 318.2-365.9 kN/m² at the northern areas.

The thematic maps indicate that the bearing capacity increases with an increase in the depth of the layers. The bearing capacity at the second layer increases by approximately 33% compared to that of the first layer, while the bearing capacity at the third layer increases by about 31% compared to that of the layer above. Overall, the bearing capacity of the east side of the study area is lower than the other zones due to low ranges of gravel content about 0-16 % and sand content by 2-14 % and high fines content ranges 65-82 %. Moreover, in the north-west zone of the depth of 0.5-1.5 m, low bearing capacity achieved as shown in Figure 7a, due to high liquid limit rage 45-57 % and plasticity index ratios range 20-27 % while the gravel percent of same area is at its range 52-76 %, Also, the same phenomena can also be noticed in the 3.0-5.0 m depth.
5. Conclusions
This research focuses on creating digital geotechnical maps by using ArcGIS software and natural neighboring interpolation. The investigation relies on data of 116 boreholes distributed over an area of 13.02 km² as the study area from geotechnical reports collected from 26 sites in Ranya city, Iraq. These are main conclusions:

1) GIS is a useful tool for capturing, displaying and analyzing geographically referenced data.
2) The maps show that with an increase in depth of the layer in Ranya soil, average gravel percentage increases by 59% in the second layer, and approximately doubled in the third layer by 89.6% compared to the first layer. Also average fines content decreases in the second and third layers by about 25.7% and 32.2% lower than the first layer, respectively while the sand content remains nearly the same.
3) Liquid limit decreased according to the first layer about 12% and 28% in the second and third layer, respectively. Plasticity index in the second layer almost remain the same and in the third layer reduces 32% compared to the first layer. Based on the maps for LL and PI, the liquid limit and plasticity index reduce from the north to south with an increase in the depth of the third layer. The highest liquid limit value of 49.9-57.4 % is observed in the second layer. However, the lowest liquid limit value of 17.5-24.5 % is observed at the south-east of the study area at the same depth. Plasticity index is similar to LL and declined from the north to the south. The lowest percentage of PI of 3.9-7.3 % is noticed in the second layer which is in the south. However, the highest percentage of PI of 26.0-29.1 % is observed at the same depth in the north part of the studied area.
4) According to the created maps for bearing capacity, high bearing capacity areas are mostly located in the southern and northern area of Ranya. However, the eastern area has the minimum bearing capacity which gradually increases towards the west.
5) The thematic maps indicate the fact that bearing capacity increased with depth. The value in the second depth increased by approximately 33% and the value in the third depth increased by 54% in comparison to the first layer. The low bearing capacity in northwest zone of the area is achieved due to the high consistency limits, and the high percentage of gravel content which ranges between 52.5-65.6%, and the same phenomena can also be noticed in the third layer.

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References


Performance of Unreinforced Hollow-Block Masonry Houses During 23 August 2017 Ranya Earthquake

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Abstract
The earthquake in Ranya City took place at local time 16:42 (GMT+3) on August 23rd 2017, with a magnitude of 5.3 Richter scale in the Kurdistan Region of Iraq. The earthquake with an approximate duration of 5 seconds occurred near Lake Dokan with a depth of 10 km below the surface of the earth. It caused a lot of damages to the structures in the region. The most vulnerable buildings were unreinforced hollow-block masonry houses which are composed of hollow-concrete blocks. This paper discusses the performance of unreinforced masonry houses of Ranya City with illustrative photos taken during on-site investigation for a number of damaged houses subjected to seismic actions. The main structural deficiencies that caused the wall cracks were highlighted, such as; very low tensile and shear resistance of the walls, large openings and their positioning, weak mortar and binding between the masonry units, existing weak joints between the crossing walls.

Keywords: Unreinforced Masonry Houses, Earthquake Damages, Hollow-Concrete Block, Seismic Performance, Ranya Earthquake.

1. Introduction
Construction is an ancient human activity that began with the essential requirement of providing an appropriate environment for daily human life. During the primary era of civilization, mud blocks and stone were commonly used for building their habitats since these natural materials were attainable in their own living places (Avila et al., 2012). The earliest forms of the construction process were load-bearing walls. Masonry block or brick wall is still one of the most common building materials with mortar. Due to the masonry wall having low tensile strength, the newer and older forms of masonry structures are extremely susceptible to earthquakes (Khadka, 2013).

Generally, in today’s construction industry process in the region, the most common type of housing is unreinforced masonry building, which mostly uses hollow concrete block units. It consists of a wall footing, unreinforced hollow block walls, lintels and reinforced slabs. The main reasons that have encouraged citizens of this region to use hollow concrete blocks; are the availability of raw materials, the rivers providing necessary sand and gravel (Ibrahim et al., 2016), and their characteristics such as durability, fire resistance and low cost. The performance of this type of structure during earthquakes mainly relies on the basic materials, which are mortar and masonry units.
Doğangün et al. (2008) reported that even in countries where there is much concern about earthquakes, most of the research had been focused on studying complex structures such as high-rise buildings, while little attention has been given to masonry houses. Additionally, observed damages on masonry buildings were very few compared to investigations on reinforced concrete buildings.

It can be noted that from an earthquake point of view, unreinforced masonry buildings are the most dangerous types of construction. Ordinarily, the walls are load-bearing and the masonry units are put on top of each other and joined together with mortar. Then the roof is built on top of the walls and its weight is transferred to the foundation through the walls. During earthquakes, inertial forces in higher amounts apply to the house at every position on the building walls and the floors (Korkmaz et al., 2010). These forces are transferred through the elements of the house, i.e., roof, wall and foundation; it can be seen that the most susceptible element to damage is the walls. As a result, the walls tremble and tip over and different types of cracks appear on the walls. Bruneau (1994) and Magenes (2006) noted that the appropriate seismic behavior of masonry buildings relies on the shear resisting mechanisms and the adequate connection between intersecting walls and between walls and floors and ceilings. Its large mass and low tensile strength of the materials is the main and most influential factors for the brittle behavior of this type of constructive system. It can be observed that for such building construction types, a number of failure mechanisms and collapses have already been distinguished when exposed to seismic forces. It is supposed that with a proper design and construction based on gained experiences registered in previous earthquakes the construction system with masonry materials can be developed to satisfy both the safety and acceptable quality conditions for constructing residential buildings (Avila et al., 2012).

![Figure 3. The Map Shows the Epicenter and The Dwellings at Risk for Damage (Earthquake-Report, 2018).](image-url)

This paper investigates the behavior of unreinforced masonry buildings and their responses to the earthquake that occurred on 23rd August 2017 in Ranya with a magnitude of 5.3 at the epicenter. The earthquake hit the north of Iraq
below or near Lake Dokan with a depth of 10 km and the shaking lasted around 5 seconds (see Figure 1) (Earthquake-Report, 2018).

2. The Characteristics of Unreinforced Masonry Houses

The main elements of the unreinforced masonry system used in the construction of residential houses are wall footing, unreinforced hollow block walls, lintels, and reinforced slabs with parapets (see Figure 2). In this section, these features will be described in detail.

2.1. Wall Footings

It is reinforced concrete footing along the whole bearing walls, one or two layers of steel bars with stirrups depending on the bearing capacity of the soil and the number of floors of the building. A typical dimension for the footing is about (50 cm – 80 cm) in width and (40 cm – 50 cm) in thickness. The majority of the houses in the study area have been constructed on strip footing without a proper foundation design process.

2.2. Load-Bearing Walls

The masonry unit that is used in this type of structure is a hollow concrete block. The raw materials and local factories for manufacturing this block type are available in the region. In the building market of the northern part of Iraq, different face sizes (Length, width, height) of blocks are produced. The old Iraqi-Standard-Specification (1987) sets detailed information concerning the dimension, category and physical requirements of the blocks. In general, Class A block is recommended to be used for load-bearing walls internally and externally. The maximum threshold of × 3 mm is allowed in the face size variations in any dimension and the minimum compressive strength of 10 N/mm² as an average value for solid and hollow concrete blocks is a must-have physical requirement (Siram, 2012). However, (Rostam et al., 2016) in their major study identified several deficiencies in dimensions, density, mixtures, production methods and strength of the concrete blocks in the regional building market and these shortcomings do not meet the Iraqi-Standard-Specification (1987). Length of 400 mm, 200 mm width and 200 mm height is the most common size for normal load-bearing walls. The block size with a length of 300 mm, width of 300 mm and height of 200 mm is usually employed under the ground and above the wall footing until the height of the damp proofing course (DPC). This reduces the bearing stress from the walls to the wall footing. Another block size with a different dimension of (Length 400 mm, width 100 mm and height 200 mm) is used for parapets and partition walls; this is mainly utilized to decrease the dead load on the slab and provide more space in the house. A mortar mix of (1:3) cement-sand (C:S) ratio to bind building blocks with 15–20 mm thickness is usually used.

2.3. Lintels

For supporting the walls upon the door and window spaces, simply supported reinforced concrete beams are used with different heights (200, 300 or 400 mm) and the width depends on the wall width.

2.4. Roof

Reinforced concrete slabs with 150-200 mm thickness are generally utilized to cover the whole structure with no joints. The concrete used for slab units usually has a compressive strength of 21 to 25MPa.
3. Common Observed Cracks and Damages

As explained before, the earthquake with a magnitude of 5.3 is moderate and can induce different levels of damage to poorly constructed buildings. Unreinforced masonry houses were damaged and cracked the most compared to other steel and concrete frame structures. Therefore, this study performs an in-depth analysis of the collocated data regarding the damaged houses in the area hit by the earthquake. It discovers the most noticeable reasons behind the occurrence of the cracks and failures in the unreinforced masonry buildings. After a thorough investigation of the damaged houses, the following dominant cracks and damages have been detected due to the earthquake.

It should be made clear that different types of cracks and damage in the unreinforced masonry walls are mainly the product of generating extreme tensile and shear stresses in the load-bearing walls. During earthquakes, in-plane and out-of-plane failures are the two most likely modes to which unreinforced masonry walls are generally exposed. The former is represented by the diagonal tensile cracks and the latter is described mainly by cracks that occur along the joints of the masonry units (Saatcioglu et al., 2005).

From the observations in Figure 3, it is apparent that the diagonal tension forces primarily cause these types of cracks, as shown in Figure 4, a schematic diagram showing the mechanism of cracks at corners. These cracks pass through the mortar joints or diagonally through the masonry units. They mostly begin at the corners of the windows and doors.
During extensive ground motion, this type of failure considerably damages the building and in extreme cases, it causes a collapse in the building.

Figure 3. Most Common Cracks in Bearing Walls Due to Bending and Shear.

- **a. Cracking of spandrel wall between openings**
- **b. Cracks at the corners of openings (door)**
- **c. Diagonal stepped shear cracking**
- **d. Cracks at the corners of openings (window)**
- **e. Double diagonal shear cracks (x shape)**
In-plane damage and cracks are produced excessively due to the intense long ground motions. This is mainly dependent on the aspect ratio of the wall, which in turn generates excessive shear forces. If this ratio is moderate, tension cracks appear on the walls diagonally (Decanini et al., 2004). During the earthquake reversal of loading, the cracks become double diagonal shear cracks that have an X shape (see Figure 3e). A study investigating the seismic risk of unreinforced masonry buildings (Erbay, 2004) reported that the cracks become stair-stepped diagonal cracks and horizontal flexural cracks in the central part of the walls (see Figure 5). These damage patterns are typical to walls that run parallel with the direction of shaking. Due to their orientation, these walls provide the lateral load resistance of the building and undergo in-plane deformation and stresses.
Pindoria et al. (2001) revealed that unreinforced masonry walls with small and few windows and doors experience less deterioration during earthquakes. Moreover, the earthquake resistance of the walls increases where the openings are far away from the corners. On the other hand, shear failures may occur to the walls with several numbers of openings. This concludes that the resistance of an unreinforced masonry building is considerably reliant upon the position, number and extent of the openings (see Figure 6).

It is clear that all directions of a building undergo the earthquake effects at the same time. The load-bearing walls orthogonal to the direction of motion are imposed to seismic inertia forces and the wall may fail in bending. As a result,
vertical tension cracks initiate at the corners, ends and center of the walls. Simultaneously, the separation of intersecting walls at the corners becomes another mode of failure due to poor connection at the wall junctions. Figure 7 shows some of the damage and out-of-plane vertical cracks that occurred to the unreinforced masonry concrete hollow block walls.

As described previously, a reinforced concrete slab directly rests on the walls without any type of connection between the walls and the slab. The top end of the walls is free to move under seismic loading. This, in turn, raises the possibility of occurring out-of-plane failure. It has been noticed that those houses having a bond beam that extends over the entire walls provide a diaphragm and they performed well with micro-cracks appeared on the surface of the walls.

The out-of-plane bending generated by the perpendicular inertial forces on the walls is the primary cause of higher tensile stresses at the corners and this induces vertical cracks.

The causes of the aforementioned damage and failures are mainly due to: (1) lack of vertical confining elements which absorb the seismic forces and increase the lateral resistance of the walls at the jambs of the openings, ends of load-bearing masonry walls, and interesting of walls; (2) the construction materials used in bed joints which are very weak in resisting tensional forces; (3) the quality of mortar which plays a pivotal role in binding the masonry units; (4) the position, size and number of openings in the walls; (5) lack of appropriate connections at the junctions of load-bearing walls; (6) the aspect ratio of the wall; (7) irregularities in the plan of the residential houses.

4. Conclusions
The 5.3 magnitude earthquake that stroked Ranya City on the 23rd of August 2017 damaged several unreinforced masonry structures. As mentioned earlier, the residential houses of this area were built by using hollow-block masonry units and mortar. The most failure mechanisms that occurred to the hollow-block masonry buildings were in-plane, out-of-plane and connection failures. Based on the damages observed and discussed, the following conclusions can be drawn:
The irregularity of the house plans and the unequal distribution of the walls in both directions made possible the out-of-plane failure and those walls underwent severe seismic forces and were damaged the most.

1) It was observed that the lateral resistance of the walls at the sides of openings, wall ends and crossing walls were insufficient. This caused the separation of the wall junctions and vertical cracks at corners at these locations.

2) It was noticed that the number, size and location of the openings in the houses had a significant impact on the performance of unreinforced hollow-block masonry houses during earthquakes. In most of the houses, openings substantially contributed to the level of damage. Most of the cracks occurred at the corners of the openings and the wall portion between the two openings.

3) The connection between the crossing walls was not appropriate and some load-bearing walls were separated with the application of seismic forces.

4) This study confirms previous findings of the performance of unreinforced masonry houses during earthquakes and contributes additional evidence that suggests the use of more proper techniques and methods to increase the lateral resistance of this type of structure.

5) Even though the earthquake was moderate, the damages and cracks were extensive. Therefore, it is recommended to increase the public awareness and workforces to construct according to standards and integrate earthquake-resistant techniques in unreinforced masonry houses.

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References


