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Research Paper

Engineering Geological Investigation of Jimma Town for Engineering Practice

Mekdes Habtamu[1], Tola Garo[2*], Yadeta C. Chemeda[2]

[1]*College of Natural and Computational Science, Bonga University, Bonga, Ethiopia*

[2]*Department of Applied Geology, School of Applied Sciences, Adama Science and Technology University, P.O. Box: 1888, Adama, Ethiopia*

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Abstract

Jimma is one of the major and densely populated towns in Ethiopia. It is located 348.5 km southwest of Addis Ababa. As in many towns of Ethiopia, very little is known about the soil and rock engineering properties and subsurface conditions of the town. Therefore, the present research work aimed to evaluate the geological and engineering geological condition of the town. Detailed field survey, in-situ strength test, sampling, geological mapping, and laboratory testing of samples were conducted. A total of 12 disturbed soil samples were collected from 12 test pits, which were dug manually up to 3 m depth. Geological and engineering geological maps were prepared at a scale of 1:25,000. Laboratory results revealed that the natural moisture content, liquid limit, plastic limit, plasticity index, specific gravity, and free swell of the soils range from 41.97–62.45%, 66–101%, 31–62%, 19–59%, 2.36–2.75, and 33–130% respectively. The area is dominantly covered by high plastic silt and clay soil type. Basalt, ignimbrite and tuff are the lithological rock units identified in the study area, which are classified into three major engineering geological subunits: very high Strength, high strength, and very low strength (soft) rock units. Flooding and high swelling potential of the soils are the main geological hazards that require due consideration in the study area.

1. Introduction

Jimma town is one of the densely populated and rapidly growing towns in the country. The town is mainly known for its immense coffee origin and production. With high population growth, the town is currently expanding in all directions, and numerous civil engineering structures such as single to multi-story buildings, roads, bridges, and industrial parks are under construction. However, the unplanned expansion of towns has exposed people and economic assets to the risk of natural disasters. In this aspect, Jimma town has to adjust its planning strategies regarding its territorial expansion and growing construction sector. Thus, there is a need for engineering geological investigation of the rocks and soils of the town.

Engineering geological investigation is a precondition for the economic design and safety of civil engineering structures (Bell, 2007; Kitata Adugna and Yosef Yirga, 2020). It involves understanding the index and engineering properties of the rocks and soils (Kelly, 2016; L'Heureux and Lunne, 2019) to identify geotechnical problems so that proper remedial measures can be provided (Wazoh and Mallo, 2021). In geotechnical engineering, soils and rocks are considered problematic if they are not suitable for construction without the adoption of some types of stabilization

*Corresponding author, e-mail: *tolagaro6@gmail..com*

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measures (Adejumo et al., 2012; Wazoh and Mallo, 2021). Expansive and collapsible soils are among the most common problematic soils in geotechnical engineering (Houston and Zhang, 2021). Problematic rocks are those that are characterized by low bearing capacity, high potential for sliding and differential settlement, dissolution, karstification, and subsidence (Vondráčková and Bellanová, 2015).

Expansive soils, in particular, are partially saturated plastic soil that exhibits high volume change when its environmental conditions are altered from dry to wet (Elshater et al., 2019). According to Kalantari (2012), seasonal moisture changes, percentage of fine materials, dry density, permeability, and presence of vegetation trees are responsible for the volumetric changes associated with expansive soils. Moreover, the degree of expansion depends on whether the soil mass contains active clay minerals such as montmorillonite (Sabtan, 2005). Numerous reports of expansive soil problems and their related damages to engineering structures such as the lifting of buildings, damage to basements, and building settlement have been documented in different countries (Rao et al., 2004; Calik and Sadoglu, 2014; Odunfa et al., 2018). This damage to engineering structures by expansive soils is estimated to be in billions of dollars per year (Sachim et al., 2014). In Ethiopia, expansive soils cover an appreciable part with an estimated aerial coverage of 24.7 million acres (LAI, 1971; Nebro, 2002). It is widely spread in the central part of Ethiopia following the major trunk roads like Addis-Ambo, Addis-Woliso, Addis-Debrebirhan, Addis-Gohatsion, and Addis-Modjo and also areas like Mekele, Jimma, and Gambella (Mesfun et al., 2019). Likewise expansive soils, problematic rocks can also pose several engineering geological problems to the structures founded on them. For example, sliding rock mass (i.e. rock slope instabilities) can cause significant damage to any infrastructure along its paths (Pantelidis, 2009). Significant variation in strength and/or bearing capacity of rocks due to geological contacts and discontinuities can lead to differential settlement of structures (Shaunik and Singh, 2020; Galindo et al., 2020), etc. Hence, to prevent damage to civil engineering infrastructures due to such problematic soils and rocks, engineering geological investigation is very crucial.

In Jimma town, however, despite recent rapid urbanization and construction activity, only a handful of studies have been conducted to avail engineering geological condition of the town. For example, Sorsa et al. (2020) and Jibril (2014) studied the geotechnical properties of soils of the town. Nevertheless, engineering geological characterization of rocks, geo-hazard and groundwater condition of the town, and their influence on construction have not been studied in the above-mentioned works. Mengesha et al. (1996) also mapped the regional geology of the Jimma area, and yet, this work has forwarded very limited engineering geological information as the study was conducted at a small scale. Therefore, this research work aimed to assess and evaluate the engineering geological condition of Jimma town to produce geological and a multipurpose engineering geological map that can be used for future land use planning, design, construction, and maintenance of engineering structures.

2. Methodology

2.1. Description of the Study area

The study area, Jimma town, is located in the south-western part of Oromia National Regional State (Figure 1). It is situated at 348.5 km from the capital, Addis Ababa, and covers an area of 50.52 square km. It lies between 846007 to 855007 m N Latitude and 258036 to 267036 m E Longitude. Elevation within the town boundary ranges from 1720 m.a.s.l. at the Kitto airfield to 2010 m.a.s.l. around Jiren Mountain. Climatically it is classified as ‘‘Weina Dega’’ with a mean daily temperature of 19.8°C and monthly rainfall ranging between 1.8 mm in February to 335 mm in June (EMA, 1981; UIDP, 2013)

2.2. Desk study

A detailed literature review on available geological and geotechnical data of the study area was carried out to gain a preliminary understanding of the engineering geological condition of the study area. These pre-existing data include topographic maps, aerial photography, satellite imagery, Digital Elevation Model (DEM), hydrogeological and geological maps. Moreover, a detailed review of the subject matter related to the research topic was also carried out. Later conceptual framework and general methodology were

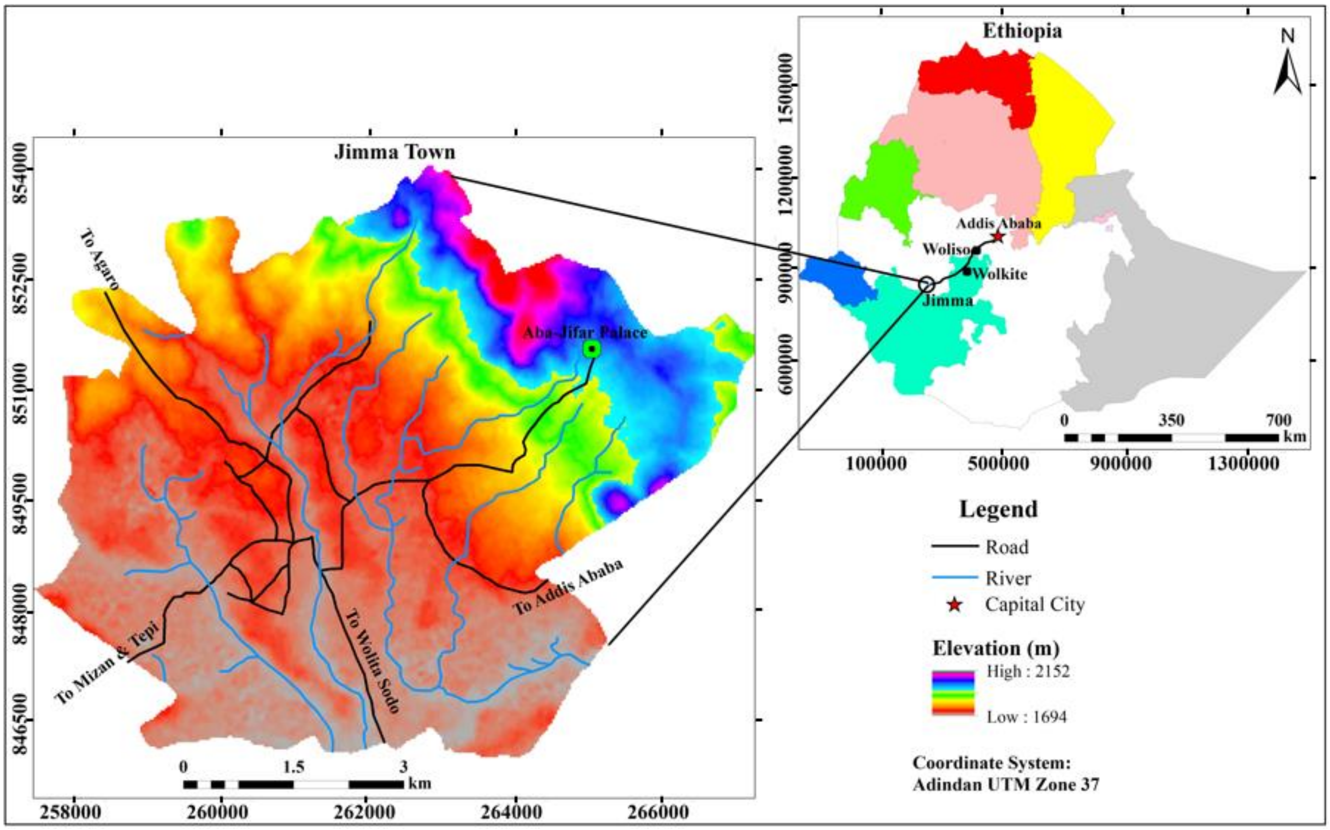

Figure 1: The Location map of the study area.

developed from the knowledge acquired through literature review and fieldwork was planned accordingly.

2.3. Field Survey and Sampling

Geological mapping, discontinuity surveying, in-situ strength testing, and soil and rock sampling were carried out in this stage. Geological mapping was carried out along systematically selected traverse lines. Descriptions of geological materials were carried out in the field in terms of color, orientation, degree of weathering, thickness, etc. The degree of weathering was determined by considering the response of rocks to blow by geological hammer following the guidelines forwarded by ISRM (1978). Discontinuity surveying was also carried out along with geological mapping to obtain parameters of discontinuity such as orientation, infill material, spacing, aperture, etc. Delineation of potential areas that are highly susceptible to geological hazards such as flooding was also carried out.

UCS of intact rock is the most important property of intact rocks that can be used in the design of civil engineering structures. It can be determined either directly at the field via Schmidt hammer or in the laboratory from representative samples (Hoek and Bray, 1981). According to Barton and Bandis (1990), strength tests in the laboratory depend on the size and how the sample is held. Schmidt hammer strength test in the field is simple, suitable, and reliable due to practically unlimited sample sizes. Based on this assumption, this study determined UCS of intact rocks of the study area directly at the field using Schmidt hammer. The guideline for the Schmidt hammer strength test was given by ISRM (1978) and ASTM (2018) in which the former recommended the use of L-Schmidt hammer type while the latter did not specify the hammer type. In this regard, Aydin (2009) suggested that N-type hammers are less sensitive to surface irregularities and are preferred for field surveys whereas L-type hammers are more sensitive to lower ranges and give a better result for weak, weathered, and porous rocks. Because of the surface irregularities and nature of rocks of the study area, this study has employed an N-type Silver Schmidt hammer using the guidelines of ASTM C805 (2018).

Accordingly, this test was conducted at 8 different locations (Figure 2), and numerous readings were taken at each location. The tests were conducted by gradually pushing the hammer perpendicular to the test surface and then corresponding hammer rebound values of rocks were recorded. Ten readings were taken at each test section with no two test locations closer together than 25 mm. A representative rebound value was obtained by discarding the rebound values that differ from the average of 10 readings by more than 7 units and then by determining the average values of the remaining readings. Finally, the representative rebound values were converted into UCS of rocks using the empirical equation suggested by Barton and Choubey (1977).

A total of twelve test pits (Figure 2) were excavated to a depth of 3 m, and disturbed soil samples were collected. The sampling locations were initially identified based on the distribution of soil in the town. Immediately after sampling, the soil samples were kept in plastic to preserve their natural moisture content.

2.4. Laboratory Test

To identify and evaluate problematic soils such as expansive soils, different laboratory tests can be conducted. For example, tests such as moisture content, specific gravity, grain size analysis, Atterberg Limits, shrinkage limits, mineralogical tests such as X-ray diffraction, swell tests, and suction measurement are recommended by several researchers such as Asuri and Keshavamurthy (2016), Elshater et al. (2019), Wazoh and Mallo (2021), etc. for identification of soil type and presence of expansive soil. In light of this concept, tests such as moisture content, specific gravity, free swell, grain size analysis, and Atterberg limits were conducted in this study to achieve the aims of the study. All of these tests were conducted at the Geotechnical laboratory of Jimma University. The tests were conducted following ASTM standards as presented in Table 1.

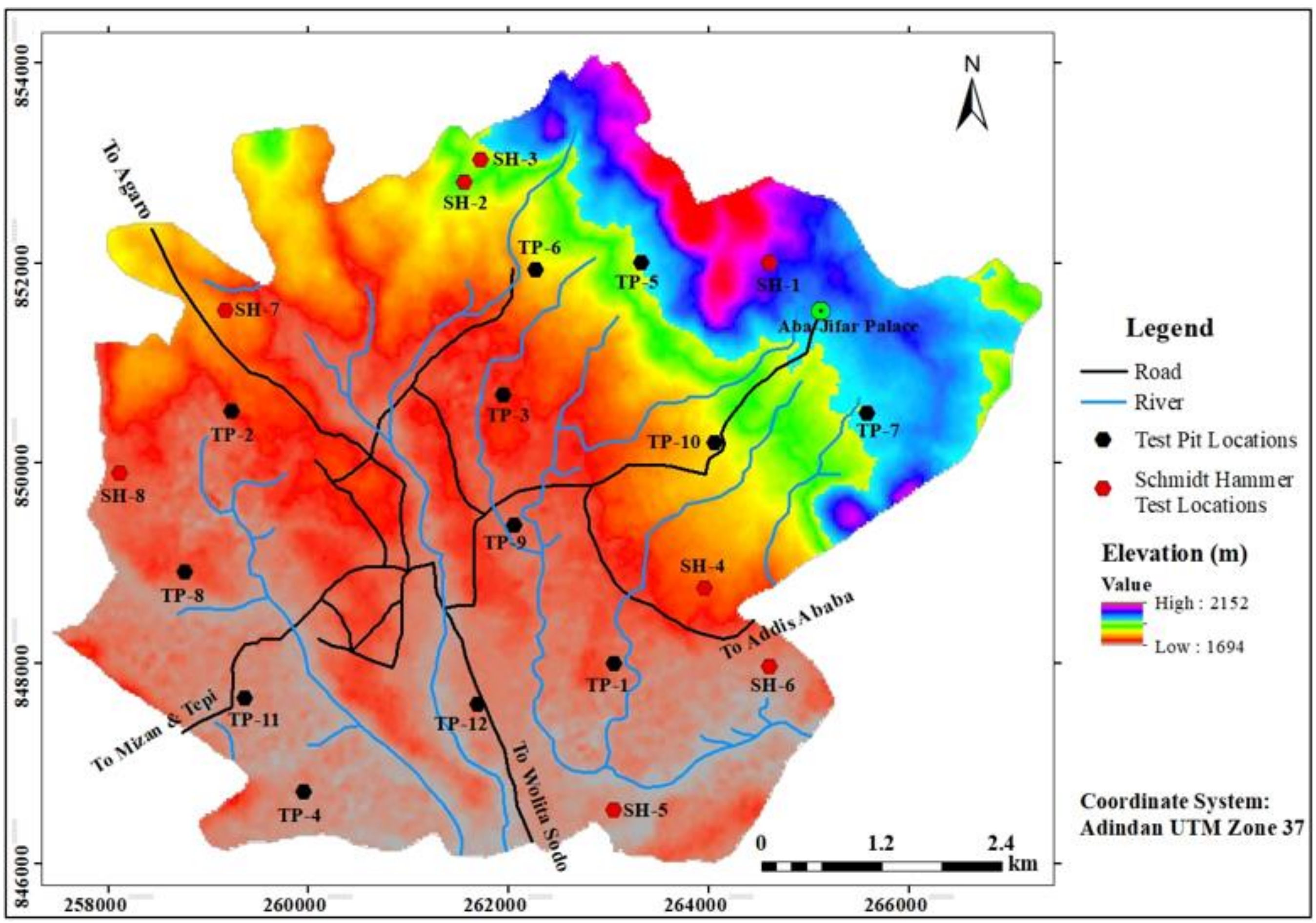

Figure 2: The location of test pits and schmidt hammer test

Table 1: Type of test and standard employed for each type of soil laboratory tests

No	Type of test	Number of Samples	Standard employed
1	Moisture Content	12	ASTM D 2216 (1998)
2	Specific Gravity	10	ASTM D 854 (1998)
3	Grain-Size Analysis	12	ASTM D 422 (1998)
4	Atterberg Limit	12	ASTM D 4318 (1998)
5	Free Swell	7	ASTM D 4546 (1998)

3. Results and discussion

3.1. Geology of the Study area

The study area is covered by quaternary soil deposits and different types of extrusive igneous rocks such as basalt, ignimbrite, and tuff units.

3.1.1. Quaternary Deposit

Quaternary soil deposits of the study area include alluvial and residual soils and cover a majority of the study area (Figure 3). The alluvial deposit is mainly exposed following the central part of the study area and extends north-south following the stream valleys. It is fine-grained, grey, and reddish in color. Residual soils of the study area are derived from in-situ weathering of basaltic and ignimbrite rocks. This deposit relatively covers a large area along gently sloped topography. The thickness of this unit is greater than 3 m as it is observed along the road and stream cut exposure.

3.1.2. Basalt

This rock unit is mainly exposed forming cliffs toward the northern and north-eastern part of the study area (Figure 3). It is fine-grained, light gray, and has a vesicular texture. It is slightly weathered and dominantly affected by horizontal and vertical joints. Some of the joints and vesicles of this rock are also filled with secondary materials such as quartz.

3.1.3. Ignimbrite

This rock unit is mainly exposed along the eastern and north-western parts of the study area. It is slightly to moderately weathered and fractured and recognized by its dark grey color. In the study area, this rock is currently being used as masonry stone to build different types of civil engineering structures due to its workability.

3.1.4. Tuff

Tuff rocks unit, which is a type of rock made of volcanic ash ejected from a vent during a volcanic eruption, is exposed in the eastern part of the study area (Figure 3). It is characterized by white color and a fine-grained texture. Due to the poor compaction and cementation of volcanic ash, this rock unit is weak and soft. In some areas, it is also found intercalated with basaltic rock.

3.2. Geological Structures

Geological structures affect the stability of civil engineering structures constructed on discontinuous rock masses. The main geological structures observed in the study area are joints. It is observed in all rock units. Most of the joints are vertically dipping with variable spacing, aperture, infill material, persistence, wall weathering, and roughness. The analysis of orientation data of joints using dips software showed that the rocks of the study area are dominantly affected by two major joint sets oriented in NE and NW, and one minor joint set oriented in N-S direction (Figure 4). In addition, one regional fault is identified in the NE part of the study area from satellite imagery and regional geological map. This fault cuts through basaltic rock and is oriented relatively in the NW direction (Figure 3).

3.3. Geological Hazard

Geological hazards can pose severe constraints on development. However, if the area prone to hazards is identified in advance, it will be very easy for the local government units to establish controlling or mitigating measures to lessen the effect of such hazards. Therefore, establishing land-use zoning considering geo-hazards is very important to protect people and their properties.

3.3.1. Flood

Jimma town is surrounded by steeply sloped ridges or hilly topographic features which is why it is frequently subjected to flooding during the rainy season. Among 13 kebeles of Jimma town, Ginjo Guduro kebele is located in a lowly elevated area and is directly affected by surface runoff from Jiren Mountain. According to Tolera Abdissa & Fayera Gudu (2019), about 38.6% of the total area of this kebele is dominantly affected by the flood. In the past decade alone, flooding has destroyed several roads, bridges, and dwelling houses in different parts of the town. Hence, due consideration is very important concerning flooding during the design of civil engineering structures in the town.

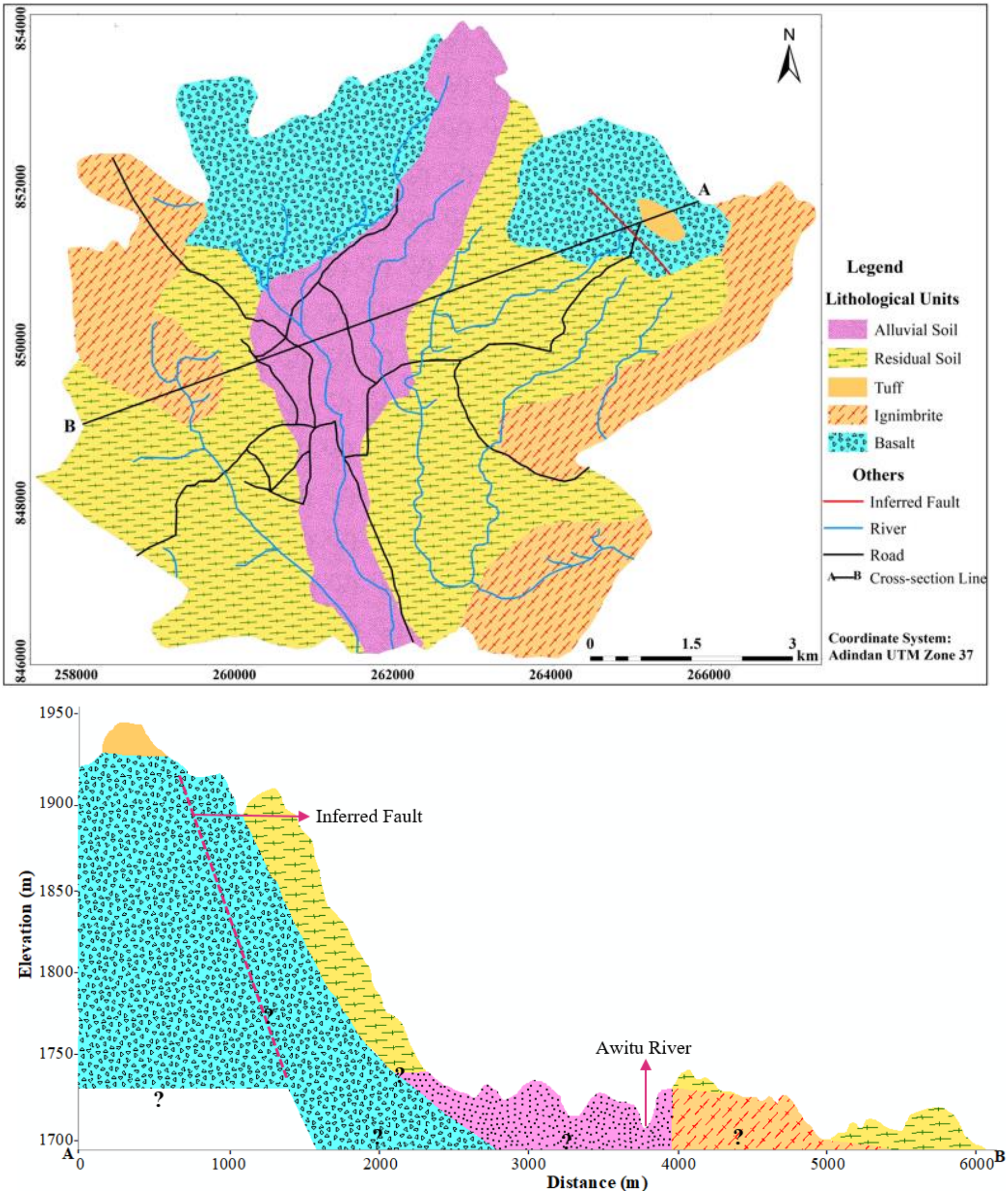

Figure 3: Geological Map (The scale is reduced from 1:25,000) and Geological Cross Section (A to B).

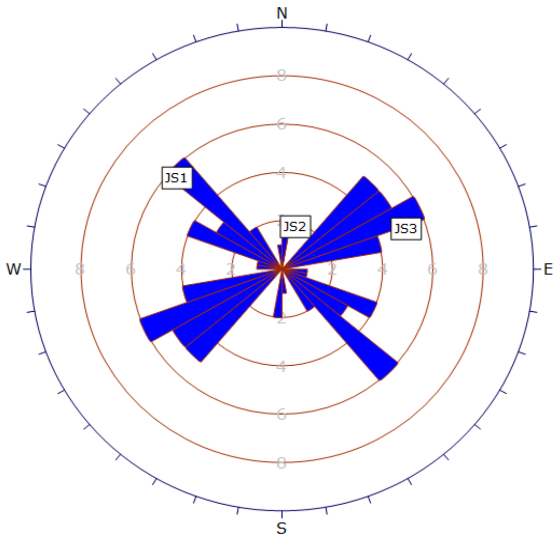

Figure 4: Rose Diagram of joints of the study area plotted using dips6.00 software.

3.3.2. Earthquake

Earthquake is shaking of the ground due to rapid release of energy and can cause catastrophic damage to anything along its path. Areas located along or close to tectonic plate boundaries are known to be highly susceptible to an earthquake. In this regard, Jimma town is located outside the main active tectonic boundary in Ethiopia, the Main Ethiopian Rift (MER). Moreover, according to the seismic hazard map of Ethiopia produced by EBCS (1995), Jimma is located in zone 1 (zone of minor damage). However, historical records have shown that the town has experienced ground shaking in the past by earthquakes that have originated from nearby areas. For example, the town has experienced ground vibration due to earthquakes that originated from Hosanna and Yirgalem area during 2010 and 2009 G.C., respectively. Moreover, an earthquake can also occur outside main tectonic plate boundaries with active regional faulting. In this aspect, Mengesha et al. (1996) have mapped some active regional normal faults in the vicinity of the study area. Hence, these faults require due consideration when evaluating the seismicity of the study area.

3.4. Index Properties and Classification of Soils

3.4.1. Moisture Content

The physical properties of most fines grained and particularly clayey soils are greatly affected by the water content. The natural moisture content of soils of the study area ranges from 41.97 to 62.45% (Table 2). The results imply that the soils of the study area dominantly have relatively high water holding capacity as the fine fraction is the dominant grain size. Hence, soils of the study area are expected to exhibit relatively higher volume change (swelling and shrinkage) associated with moisture content variation. Moreover, the high moisture content of some soils of the study area, particularly TP-3 is potentially attributed to high organic content. This is because organic composts with high organic carbon and micronutrient content impart high water holding capacity to the soil (Ram and Masto, 2014) leading to a direct relationship between the percentage of organic matter and water content (Pradeep and Vinu, 2015)

3.4.2. Specific Gravity

Specific gravity is an important parameter of soils as it is used for the determination of void ratio and particle size (Arora, 2004). The specific gravity of the soils in the study area varies from 2.36 to 2.75 (Table 2). Pradeep and Vinu (2015) stated that specific gravity non-linearly decreases with an increase in organic content as organic material is comparatively light in weight with respect to soil particles. As these light organic matters occupy a considerable portion of the unit volume of soil, highly organic soil possesses lesser specific gravity. The amount of lightweight organic matter present in compost amended organic soil yields low specific gravity values (Puppala et al., 2007). Hence, the lower specific gravity value for some soils of the study area, such as TP-3 could be due to the presence of organic matter.

3.4.3. Grain Size Analysis

Grain size analysis is used to determine the amount of different types of grain size present in the soil. The analysis was done according to the standard ASTM D 422 (1998). For soil particles greater than 75 µm in diameter, sieve analysis was done, while hydrometer analysis was performed for particles less than 75 µm in diameter. The result shows that the predominant size of soil particles in the study area is fine-grained soils which have gravel content ranging from 0-0.98%, sand fraction 0.35-4.59%, silt fraction 24.05-55.91%, and clay 42.16-71.36% (Table 2). Amagu et al. (2018) stated that the implication of a higher percentage of fines in an engineering material is that the finer soil particles can be easily eroded away by water thereby decreasing the bonding between soils and making compaction difficult. This implies that almost all the studied soils of the study

Table 2: Natural Moisture Content, Specific gravity, and Grain Size Analysis results

Sample No.	Depth (m)	NMC (%)	Specific Gravity	Grain Size Distribution			
				Gravel	Sand	Silt	Clay
TP-1	3	44.79	2.63	0.13	1.20	38.03	60.51
TP-2	3	50.52	2.55	0.03	0.62	48.08	50.95
TP-3	3	62.45	2.36	0.00	3.16	40.15	56.69
TP-4	3	52.39	2.65	0.12	2.18	35.15	62.42
TP-5	3	52.08	2.61	0.98	0.35	37.34	57.20
TP-6	3	42.18	2.59	0.00	1.21	41.59	57.20
TP-7	3	48.61	2.62	0.12	1.64	50.69	47.55
TP-8	3	51.07	2.53	0.26	2.99	52.36	44.13
TP-9	3	44.69	2.74	0.00	1.93	55.91	42.16
TP-10	3	44.82	2.75	0.00	4.59	24.05	71.36
TP-11	3	44.25		0.00	2.32	30.00	67.60
TP-12	3	41.97		0.02	3.38	41.60	54.98

area are susceptible to erosion effect and hence, flooding as well. Moreover, based on Elshater et al. (2019) and Wazoh and Mallo (2021) high percentage of fines can also imply a high potential for expansiveness.

3.4.4. Consistency Limit and Swelling Potential

The Atterberg limits test results of the studied soils are summarized in Table 3. The test results analysis showed that liquid limit, plastic limit, and plasticity index of soils in the study area range from 66%-101%, 31%-62%, and 19%-59%, respectively (Table 3). Based on the study conducted by Azam et al. (2013), Calik and Sadoglu (2014) and Christodoulia (2015), soil with a liquid limit above 54–83% are considered expansive with a greater tendency for compressibility and excessive settlement. All the liquid limit values of soils of the study area are either greater than or within the above range (Table 3) which is potential proof for the expansiveness of soils of the study area. Moreover, based on recent classification of soil forwarded by Elshater et al. (2019), with exception of soils obtained from TP-2, TP-8, and TP-11 that are medium plastic, all the soils of the study area fall into high to very high expansive/or plasticity classes. This also implies a high degree of the expansiveness of soils of the study area. Such high expansive potential makes the soils of the study area highly susceptible to differential settlement of the foundation (Wazoh and Mallo, 2021). With regard to the use of soils as sub-base and sub-grade material, Adeyemi (2002), Rao et al. (2004), etc. recommended that the plasticity index should be less than the upper limit of 25%. Based on these studies, with exception of soils obtained from TP-2, TP-8, and TP-11 (Table 3), all are unsuitable for sub-base and sub-grade foundation materials because they have a plasticity index >25%.

3.4.5. Activity

Activity is first described by Skempton (1953) and is a ratio of plasticity index of soil to the percent of clay size fraction in the soil. It is widely used to identify the type of clay mineral in the soil and estimate the swelling potential of soil. Accordingly, Skempton (1953) suggested that if the activity is less than 0.75 soil is inactive, 0.75–1.25 soil is normal, and greater than 1.25 soil is active. In the light of these concepts, the activity of the soils of the study area was computed and found to be in a range of 0.28–1.02 (Table 3). The analysis of the results also showed that 50% of studied soils of the study area are classified as normal soil while the remaining were classified as inactive soils.

3.4.6. Free Swell

Free swell is used to determine the potential of a soil for expansiveness. Accordingly, the free swell soils of the study area vary from 33–130% (Table 3). Soils with a free swell value less than 50% are considered non-expansive, 50–100% are marginally expansive, and greater than 100% are expansive (BIS IS 1498: 1970(R2002)). Based on this classification, soils that

Table 3: Atterberg limits, activity, and free swell of soils of the study area

Sample No.	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Activity	Free Swell (%)
TP-1	66	31	35	0.57	130
TP-2	70	49	21	0.41	58
TP-3	92	43	49	0.86	43
TP-4	97	38	59	0.94	100
TP-5	101	53	48	0.83	63
TP-6	87	34	53	0.92	125
TP-7	82	54	28	0.58	33
TP-8	72	53	19	0.43	
TP-9	74	42	43	1.02	
TP-10	75	45	30	0.42	
TP-11	81	62	19	0.28	
TP-12	91	37	54	0.99	

were obtained from test pits TP-1, TP-4, and TP-6 are classified as expansive soil. Similarly, soils from test pit TP-2 and TP-5 are classified as marginally expansive while those from test pit TP-3 and TP-7 are classified as non-expansive. According to Holtz and Gibbs (1956) cited in Bell (1983), soils with a free swell value as high as 100% can cause considerable damage to lightly loaded structures, and soils with a free swell value below 50% seldom exhibit appreciable volume change even under very light loadings. Therefore, careful consideration is required concerning the design of different types of structures in the parts of the study area that are characterized by free swell as high as 100 or more.

3.4.7. Engineering Classification of Soils

A soil classification scheme provides a method of identifying soils in a particular group that would likely exhibit similar characteristics and is very important for the preliminary assessment of engineering properties of soils. It is also used to specify a certain soil type that is best suited for a given application. There are several classification schemes available and were devised for specific use. For example, the American Association of State Highway and Transportation Officials (AASHTO) developed one scheme that classifies soils according to their usefulness in roads and highways. In contrast, the Unified Soil Classification System (USCS) was initially developed for airfield construction but was later modified for general use.

In this study, both AASHTO and USCS were used to classify the soils of the study area. According to the AASHTO classification system, all soil samples obtained from the study area were classified under group A-7-6 category (Table 4), and hence their suitability as sub-grade material is poor. Concerning group index in this classification system, Das (2012) suggested that the quality of performance of soil as a subgrade material is inversely proportional to the group index. Moreover, Arora (2004) forwarded that soils with group index 20 or greater show very poor sub-grade. In the light of these concepts, all soils of the study area have a group index greater than 30 indicating that they are very poor to be used as sub-grade material without treatment.

Similarly, based on USCS, soils obtained from TP-1, TP-4, TP-6, and TP-12 were classified as Inorganic clay of high plasticity (Table 4 and Figure 5). On the contrary, soils from the remaining test pits were classified as Inorganic silt of high plasticity (Table 4 and Figure 5). According to Alhassan et al. (2014), clay minerals that are present for the soils that fall in the region of high plasticity and compressibility in the plasticity chart are dominantly expansive clays. This implies that the clay minerals in the soils of the study area are dominantly expansive. In general, according to the general engineering suitability of the soils (Arora, 2003), the soils of the study area are characterized by high compressibility, fair to poor shear strength, and poor engineering workability. The results indicate that they are problematic soils for engineering application with higher swelling-shrinkage potential.

Table 4: Classification of soils of the study area based on AASHTO and USCS system.

Sample No.	AASHTO classification	USCS	
		Group Symbol	Group Name
TP-1	A-7-6(42)	CH	Inorganic clay of high plasticity
TP-2	A-7-6(32)	MH	Inorganic silt of high plasticity
TP-3	A-7-6(60)	MH	Inorganic silt of high plasticity
TP-4	A-7-6(70)	CH	Inorganic clay of high plasticity
TP-5	A-7-6(61)	MH	Inorganic silt of high plasticity
TP-6	A-7-6(63)	CH	Inorganic clay of high plasticity
TP-7	A-7-6(41)	MH	Inorganic silt of high plasticity
TP-8	A-7-6(30)	MH	Inorganic silt of high plasticity
TP-9	A-7-6(42)	MH	Inorganic silt of high plasticity
TP-10	A-7-6(39)	MH	Inorganic silt of high plasticity
TP-11	A-7-6(33)	MH	Inorganic silt of high plasticity
TP-12	A-7-6(65)	CH	Inorganic clay of high plasticity

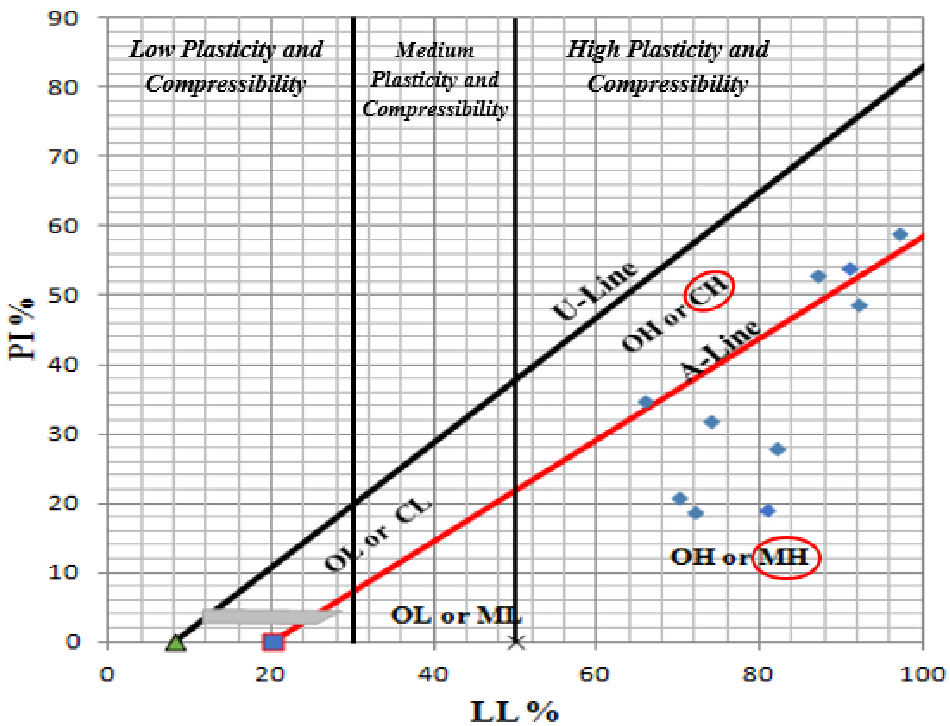

Figure 5: Classifications of Soils of the study area based on USCS.

3.5. Groundwater

Groundwater influences excavation and construction methods by flowing into excavations, producing seepage forces and uplift pressures, and causing corrosive action (UNESCO, 1976). Hence, groundwater level and quality information are necessary for construction activities. As the well data summarized in Table 5 shows, the groundwater exist at shallow depth in different parts of Jimma town with static water level as shallow as 0.31 m in Boche-bore well. Therefore, the influence of groundwater on the foundations could be critical in the town.

Groundwater level contour map (Figure 6) was also prepared from static water level data to suggest possible groundwater flow direction. From the map analysis, it is found that groundwater flow is toward the south-western direction following the topography. Hence, groundwater discharge is relatively higher in the south-western section of the town leading to a very shallow static water level. Therefore, due consideration regarding groundwater is very important in case engineering structures are intended to be constructed in this area.

3.6. Engineering Geological Characterization of Rocks

Engineering geological characterization of rocks of the study area was done via in-situ strength testing using Schmidt hammer and visual estimation of weathering grade. Accordingly, the rocks of the study area are classified based on their Unconfined Compressive strength (UCS) as per ISRM (1978) classification system. According to this classification system, rocks of the study area are classified into three major engineering geological subunits: Very high Strength, High Strength, and very low (soft) rock units.

3.6.1. Very High Strength Rock Units

These engineering geological subunits include slightly weathered ignimbrite and basalt.

Table 5: Measurement values of water level elevations of the wells

Well	Longitude (m)	Latitude(m)	Altitude (m)	Groundwater level elevation (m)	Groundwater depth (m)
Ginjo School	264841	852239	1767.97	1764.17	3.80
Kito-furdisa	263366	851619	1756.98	1753.98	3.00
JU sport field	260398	852256	1744.85	1742.65	2.20
Main campus 1	262846	849088	1737.2	1736.00	1.20
Main campus 2	263265	848836	1736.24	1735.44	0.80
Airport	259040	849892	1725.48	1724.57	0.91
Boche-bore	263734	847830	1722.47	1722.16	0.31

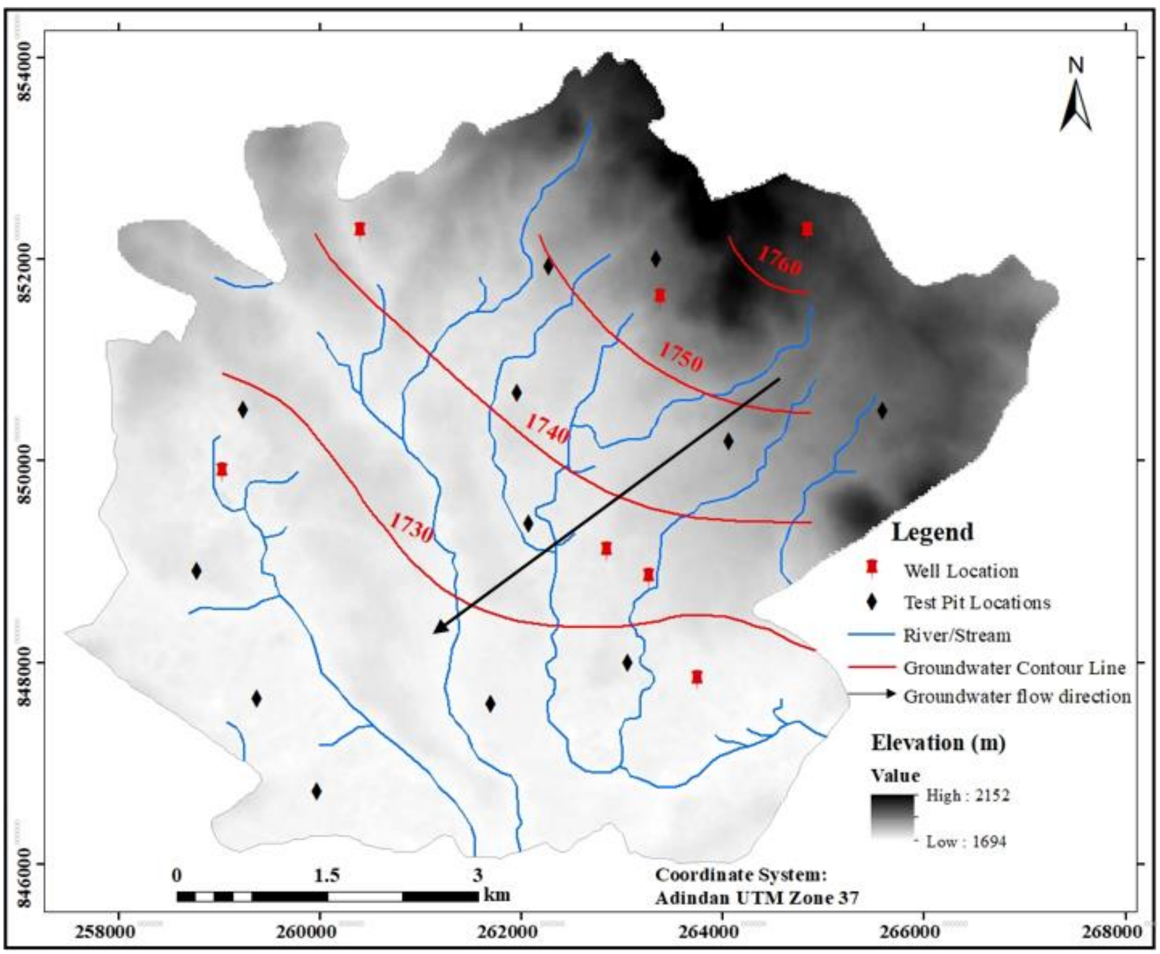

Figure 6: Groundwater level contours and groundwater flow direction of the study area

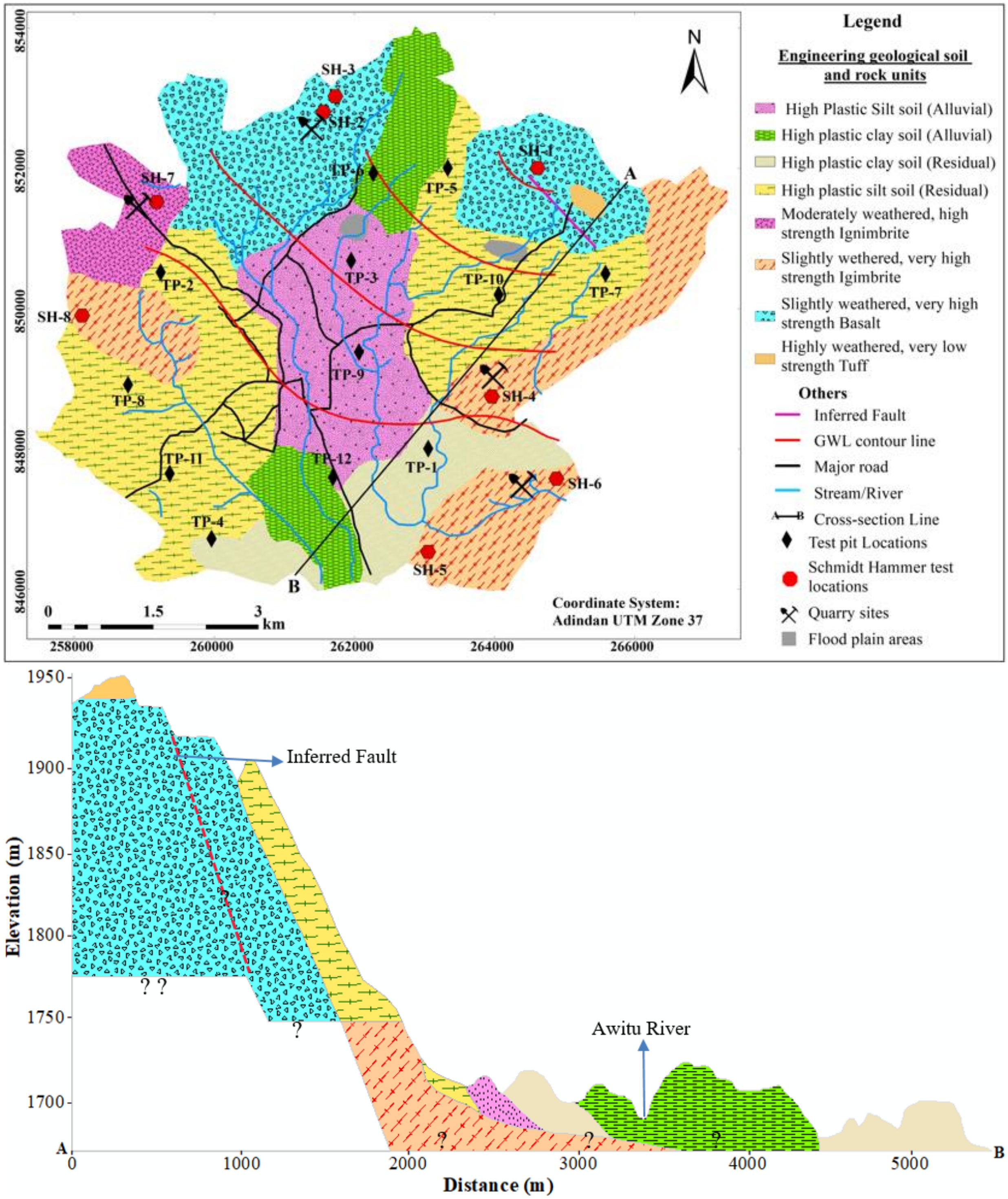

Figure 7: Engineering geological map (Scale 1:25,000) and cross-section of the study area

3.6.1.1 Slightly Weathered Very High Strength Ignimbrite

Very high strength ignimbrite rock (Figure 7) unit is found in the western, eastern, and southeastern parts of the study area forming cliffs. It is gray in color. It has porphyritic texture (i.e. clasts and groundmass) and is slightly weathered. Its UCS value varies from 131.87 to 169.82 MPa.

3.6.1.1. Slightly Weathered Very High Strength Basalt

Very high-strength basalt is mostly found at the N and NE part of the study area. It is fine-grained and slightly weathered. The strength of the rock material varies between 107.15 to 194.98 MPa. Three sets (NW, SE, and NS) joints are dominant. The joints are moderately spaced (250 mm to 300 mm). The joint surface is both smooth and rough. The aperture varies from tight to partly open (0.25 mm to 1 mm).

3.6.2 Moderately Weathered High Strength Ignimbrite

The high-strength ignimbrite rock unit is found in the NW part of the study area. It is light gray in color and moderately weathered. The strength of the rock material range from 50.18 to 97.72 MPa.

3.6.3. Highly Weathered Very Low Strength Rock Unit (Tuff)

The strength of this rock was too low to be measured using the Schmidt hammer during the field survey. The rock of this unit can be peeled with a knife, and hence the compressive strength is considered to be <1 Mpa. It is highly weathered, light in color, and has a fine-grained texture.

4. Conclusion and Recommendations

The main mappable soil units in the study area are alluvial and residual soils. From grain size analysis, the soils are dominantly fine-grained with gravel, sand, silt, and clay contents ranging from 0% to 0.98%, 0.35% to 4.59%, 24.05% to 55.91%, and 42.16% to 71.36%, respectively. The natural moisture content varies from 41.97% to 62.45% indicating that the water holding capacity of the soils is relatively very high. The liquid limit and plastic limit of soils in the study area range from 66%–101% and 31%–42%, respectively. Similarly, the plasticity index also varies from 19%-59%, showing that soils of the study are medium to very highly plastic. Their free swell potential is also in a range of 33% to 130% indicating that most soils of the study area are characterized by a high degree of expansiveness, and hence various types of structures constructed on soils of the study area are susceptible to considerable damage. According to USCS, soils of the study are classified as high plastic clay (CH) and high plastic silt (MH) and are characterized by high compressibility, fair to poor shear strength, and poor engineering workability. Likewise, as per the AASHTO classification system, the soils are classified as clayey soils (A-7-6) which is unfavorable for subgrade material. From engineering rock characterization, four engineering geological rock units were identified in the study area considering the degree of weathering and compressive strength of intact rock. These engineering geological units include slightly weathered very high strength ignimbrite, moderately weathered high strength ignimbrite, slightly weathered very high strength basalt, and very low strength highly weathered tuff units. Moreover, the study area is also highly susceptible to flooding hazards.

The design of any civil engineering structures in the town shall consider the high swelling potential of soils of this area. Construction of detention dam, drainage networks, diversion canals, and proper waste management is also recommended to minimize the flooding risk in the town.

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