# "A comparison of the Impacts of Tap and Saline Water on Some Important Engineering Properties of Virgin and Flyash mixed Swelling Soil"

*A thesis submitted in partial fulfilment of the requirements for the degree of*

**Bachelor of Technology**

*in*

**Civil Engineering**

*by*

**Karan Aswani**

Roll Number – 10601036

UNDER THE GUIDANCE OF PROF. N.R. MOHANTY

**DEPARTMENT OF CIVIL ENGINEERING**

**National Institute of Technology**

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# CERTIFICATE

This is to certify that the thesis entitled **"A comparison of the Impacts of Tap and Saline Water on Some Important Engineering Properties of Virgin and Flyash mixed Swelling Soil**" submitted by **Karan Aswani**, Roll No. 10601036, is a record of bonafide work carried out by him under my guidance.

The results presented in the thesis have not been submitted elsewhere for award of any degree.

Prof. N.R. Mohanty

Department of Civil Engineering

National Institute of Technology, Rourkela

# ACKNOWLEDGEMENT

With the deepest sense of gratitude let me express my hearty indebtedness to my revered guide Prof. N.R. Mohanty, Professor, Civil Engineering Department, NIT Rourkela for having guided me in working on this project. Without his constant supervision and keen interest in the day-to-day progress of the project, it would not have been possible for me to place this work in the present form.

I feel equally grateful to the ground-root staff of Geotechnical Engineering Laboratory for helping me without any regard to their time constraints.

Karan Aswani

Roll No. - 10601036

# INDEX

| CONTENTS | PAGE No. |
| --- | --- |
|  |

# ABSTRACT

Around 20% area of our country and also more or less the world, at large, is covered with swelling soil. Distress of structures due to swelling soil prohibits multi-storeyed buildings on such areas and as a result of this both cost of construction as well total area under a structure increases remarkably. Besides, safety of the building is also threatened.

Researchers, world over use one or the other type of admixtures to stabilize the swelling soil and restrict its swelling properties. But all the admixtures cannot have the same type of influence over the swelling soil. Besides the saturating medium may also have an impact on swelling characteristics.

In coastal areas there are long bay-roads which are very near to sea. Seasonally or during tidal surges in the sea, saline water is likely to saturate the soils of those roads. Roads may also remain partially submerged during some period of the year.

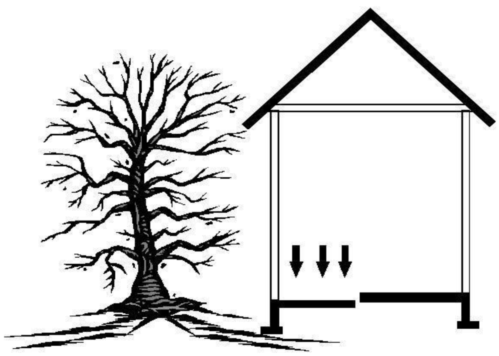
Flyash has been long utilised as an admixture in controlling swelling of expansive soils. It is therefore important to study how flyash stabilised coastal roads( in expansive soil area) behave with respect to swelling when they are saturated by saline water in contrast to that when the saturating water is ordinary water. In this regard some important engineering tests like differential free swell test and Atterberg's limit test were conducted on virgin soil and swelling soil-flyash mix saturated in tap water and then in sea water.

(Chapter 1)

# INTRODUCTION

Soil is one of the most commonly encountered material in civil engineering. All the structures except some, which are founded on solid rock, rest ultimately on soil. Geotechnical engineers all over the world face enormous problems, when the soil founding those structures are expansive in nature. This expansiveness is imparted to such soils when they contain clay minerals such as montmorilonite, illite, kaolinite etc. in appreciable quantity. It is due to them (clay mineral) that the swelling soils expand on wetting and are subjected to shrinkage on drying.

The problem of instability of structures constructed on such soil is mainly due to lifting up of the structures on heaving of soil mass (under the foundation) on saturation during rainy season and settlement due to shrinkage during summer season. Due to this cavities are formed leading to loss of contact between the soil and structures at some points. This in turn leads to splitting of structure and failure due to loss of shear strength or unequal settlement.



On the contrary, during rainy season when the foundation soil swells it is restrained by the foundation to do so. As a result, an upward swelling pressure is exerted by the soil on the foundation. As this pressure is not uniform everywhere, the net downward pressure becomes uneven. This also leads to unequal settlement, leading to progressive failure of structures.

In saline condition, as the treating of lime to swelling soil has reported to be uneconomic and ineffective, in the present investigation flyash has been chosen as a stabilizing swelling soil in coastal areas. The other reasons behind the choice of flyash as a stabilizer are as follows :

i) Flyash is costless and abundantly available all over the country.
ii) As flyash is a by-product of thermal power plants, land area required for its disposition is a great problem in a densely populated country like India.
iii) Utilization of flyash solves the problem of air and water pollution.
iv)

The object of the present investigation is to study the effect of saline water on virgin swelling soil as well as flyash mixed swelling soil. To it, was mixed flyash in different proportions by weight from 0%(virgin soil),10%,20%,30%,40%,50%,60% and 70% of flyash content.

Soil engineering tests like differential free swell test and Atterberg's limit were conducted on virgin soil and soil-flyash mix. Though it was also required to conduct shear or other strength tests in presence of water and saline water, the same could not be done due to constraints of time.

(Chapter 2)

# REVIEW OF LITERATURE

# 2.1 SWELLING OF CLAYS

It is known that montmorillonite clay minerals having expanding type of lattice structure swell upon wetting. Swelling brings with it the change in physical and mechanical properties. Potentially expansive clays are confining to the semi-arid regions of tropical and temperate climate zones. Expansive clays are in abundance where the annual evapotranspiration exceeds the precipitation.

The swelling is absent or limited in illite and kaolinite. The water having dipolar nature trying to enter the expanding lattice structure seems to be responsible for the swelling of clays. Irrespective of high swelling potential if the moisture content of the clay remains unchanged, there will be no volume change and the structure founded on clay remain unchanged, there will be no volume change and the structure founded on clay with constant moisture content will not be subjected to movement caused by heaving. Complete saturation is not necessary to accomplish swelling. A slight change of moisture content in magnitude is sufficient to cause detrimental swelling.

For swelling to start, the clay should have minimum initial moisture content. The probable initial water content( W min) from which swell will begin beneath a prepared subgrade is given by

W min%= 0.2w + 9

Where w = liquid limit of clay

Swelling behaviour of the compacted clay will be primarily governed by the following factors :

- Composition of the clay :- Composition and the amount of clay minerals, nature and amount of the exchangeable cations, proportions of sand and silt in the clay and presence of organic matter and cementing agents.

- Compaction conditions :- Moulded water content, dry density, degree of saturation and type of compaction affect the swelling characteristics.

- Chemical properties of the pore fluid, both during compaction and that which is imbibed during swelling.

- Shape,size and thickness of the sample.

- Initial placement conditions, temperature and volume change.

- Time allowed for swelling.

# 2.2 MECHANISM OF SWELLING

Swelling can be explained by osmotic repulsive pressures arising from the difference in the ion concentration in the double layer water between interacting clay particles and that on the free pore water.

# 2.3 SWELLING IN SATURATED CLAYS

When a clay particle is immersed in pure water, sufficient exchangeable cations surround the particle and are attracted to it by a net negative charge in the clay particle in such a manner that the cations plus the particle constitute an electrically neutral system. The system is designated as the clay "micelle". The ions and water within the micelle constitute the double layer. If the clay particles are immersed in the salt solution instead of pure water, the anion would also be present in the double layer, but the number of cations would be increased so that the micelle still remain electrically neutral.

# 2.4 IDENTIFICATION AND CLASSIFICATION OF SWELLING SOIL

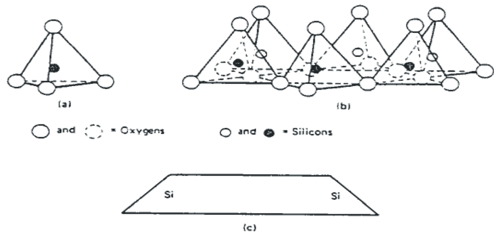
## IS CLASSIFICATION SYSTEM (IS 1498-1972)

| Liquid Limit | Plasticity Index | Shrinkage Index | Free Swell | Degree of Expansion | Degree of Severity |
|---|---|---|---|---|---|
| 20-30 | <12 | <15 | <50 | Low | Non critical |
| 35-50 | 12-23 | 15-30 | 50-100 | Medium | Marginal |
| 50-70 | 23-32 | 30-60 | 100-200 | High | Critical |
| 70-90 | >32 | >60 | >200 | Very high | Severe |

# 2.5 CLAY MINERALS

There are two fundamental building blocks for the clay mineral structures. One is a silica tetrahedral unit in which four oxygen or hydroxyls having the configuration of a tetrahedron enclose a silicon atom.

The tetrahedral are combined in a sheet structure so the oxygens of the bases of all the tetrahedra are in a common plane, and each oxygen belongs to two tetrahedral.

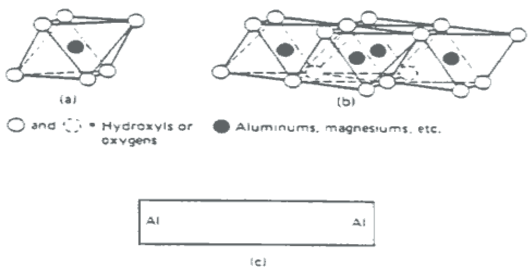



(a) Silica tetrahedron.
(b) Silica tetrahedral sheet.
(c) Schematic of silica sheet. See Table 2-1.

Source: Lambe, 1958; Grim, 1968

Figure 2-1. Clay mineral tetrahedral sheet structure.

The second building block is an octahedral unit in which an aluminium, iron or magnesium atom is enclosed in six hydroxyls having the configuration of an octahedral. The octahedral units are put together into a sheet structure which may be viewed as two layers of densely packed hydroxyls with cation between the sheets in octahedral co-ordination.




(a) Octahedron.
(b) Octahedral sheet.
(c) Schematic of gibbsite octahedral sheet. See Table 2-1.

Source: Lambe, 1958; Grim, 1963

Figure 2-2. Clay mineral octahedral sheet structure.

GROUP OF MINERALS AND THE MOST IMPORTANT MINERAL IN EACH GROUP

| S. No | Name of the minerals & group | Structural formula |
|---|---|---|
| I | Kaolin Group<br>1. Kaolinite<br>2. Hallyosite | <br>$Al_4 Si_4 O_{10} (OH)_8$<br>$Al_4 Si_4 O_6 (OH)_{16}$ |
| II | Montmorillonite Group<br>Montmorillonite | <br>$Al_4 Si_8 O_{20} (OH)_4 nH_2O$ |
| III | Illite Group<br>Illite | <br>$Si_{8-y} Al_y (OH)_4 O_{20}$ |

# KAOLIN GROUP

Kaolin Group of minerals are the most stable of the group of minerals. Kaolinite is found by the stacking of the crystalline layers of 7 A thick, one above the other with the base of the silica sheet bonding hydroxyls of gibbisite sheet by hydrogen bond. Since hydrogen bonds are comparatively stronger, Koalinite crystals consist of many sheet stacking, difficult to dislodge. Thus the mineral is stable and water cannot enter between the sheets to expand the unit cells.

# MONTMORILLONITE GROUP

Montmorillonite minerals form weak bondage between each other and an excellent cleavage between them. As a result, soil containing considerable amount of montmorillonite minerals exhibit high swelling and shrinkage characteristics.

Montmorillonite is the most common of the group of minerals. Structural arrangement of this mineral is composed of units made of two silica tetrahedral sheets with a central octahedral alumina sheet. The silica and gibbisite sheets are combined in such a way that the tips of the tetrahedrons of each silica sheet and one of the hydroxyl layers of the octahedral sheet form a common layer. The atoms common to both the silica and gibbisite layers become oxygen instead of hydroxyls. The thickness of the silica-gibbisite-silica unit is about 1 A.

In stacking of these combined units one above another, oxygen layers of each unit are adjacent to oxygen of the neighbouring units, with a consequence that there is a weak bond(mainly due to Vander waal's forces) and an excellent cleavage between them. Water can enter between the sheets causing them to expand significantly and thus structure can break into 10 A thick structure units. Bentonite clay and Black cotton soil belong to the montmorillonite group.

Exchangeable ions can easily enter between the layers causing the layers to be separated. Because of affinity for water, clay soils containing montmorillonite mineral are susceptible to substantial volume changes. They swell as the water gets entered into the lattice structure and shrink if the water is removed because of the same reason. In a moist state, montmorillonite is highly plastic and has little internal friction. Its excessive swelling capacity may seriously endanger the stability of overlying structures and road pavements.

# ILLITE GROUP

Structural unit of illite mineral is similar to that of montmorillonite except that some of the silicons are always replaced by aluminium atoms and the resultant charge deficiency is balanced by potassium ions. The bonds with the non-exchangeable potassium ions are weaker than the hydrogen bonds, but stronger than the water bond of montmorillonite.

# ABSORBED WATER

The layer of water, which surround the clay crystals, are called adsorbed water. Firstly, water molecule being a dipole, is electro statically attracted attracted to the negatively charged surface of the crystal. Secondly, water is being absorbed by hydrogen bonding i.e. hydrogen atom of water molecule is attracted to the oxygen or hydroxyls on the surface of the clay particles. It seems natural that the attraction of water to the clay particles is very strong near the surface and tapers off with distance. The water dipoles are also strongly oriented at the surface. More than one layer of water molecules are adsorbed and the degree of orientation decreases as the distance increases. The highly oriented water is considered to effect the behaviour of clay particles when subjected to stresses, since the adsorbed film of water comes between the particle surfaces.

## 2.6 PROBLEMS ASSOCIATED WITH SWELLING SOILS

Swelling and shrinkage are generally confined to the upper portions of a soil deposit essentially caused by seasonal moisture variations. The same swelling soil which depicts very low shear strength and bearing capacities under swelling due to imbibing of water during rainy season shows remarkably high bearing capacity after shrinkage consequent on drying during summer.

As in the case of typical clayey soil, in the expansive soil, shrinkage cracks of the order 15cm width travelling about 9 to 10 feet deep is a common sight in the field where such deposits exists. Construction of extended projects like railways and roads in such areas always face serious problems of an extremely low bearing capacity of the subgrade when it is wet and also its high swelling and swelling pressure characteristics during the process of wetting and threat of facing shrinkage cracks. Tracks and roads built on such soil without proper measures face distress and need constant and costly maintenance even for average performance.

## 2.7 REMEDIAL MEASURES FOR BUILDING STRUCTURES ON EXPANSIVE SOIL

Depending upon the use of swelling soils as foundation support or construction materials, their properties need careful studies to estimate their potential for damages based on volume change with reference to the imposed structural loads and the tolerance of structures for maximum settlement, differential settlement etc. Considering the requirement of the structural tolerance required, proper remedial measures are to be thought of for controlling the effect of expansive properties of the soil on the structure.

There are number of measures that can be utilised to minimise or eliminate the damage for specific structures associated with the expansive soils are mentioned below :

- INSULATION

  Prevention of moisture penetration into the foundation soil is obtained by providing different types of impervious aprons around the building. Provision to absorb soil movements occurring below the foundation is obtained by partially filling the foundation trenches with materials like rubber or sand.

- UNDER REAMED PILE FOUNDATION

When the foundation soil consists of expansive soil, the bulb of the under-reamed pile provides an anchorage against uplift due to swelling pressure apart from the increased bearing.

- CAST IN SITU PILE AND BEAM FOUNDATONS

The principle involved is similar to that of under-reamed pile foundation,except that additional load bearing capacity and the necessary resistance to uplift forces are obtained in this method by an extra depth of straight pile stem which replaces the under-reamed base.

- PRE-WETTING

Pre-wetting theory is based on the assumptions that is the soil is allowed to swell prior to construction and if the high soil moisture content is maintained, the sol volume will remain essentially constant, achieving no heave state and therefore structural damage will not occur.

- MOISTURE BARRIERS

Moisture barriers may be constructed around the perimeter of slab-on-grade to minimise the moisture variation beneath the perimeter of slab.

- COMPACTION

One of the most methods of controlling heave or expansive soil is to compact it at moisture content and unit weight which will minimise expansion.

- SAND REPLACEMENT

Partial or complete replacement of swelling soil by sand serves as a method to minimise the swelling effect.

- COHESIVE NON-SWELLING SOIL LAYER

By providing adequate thickness of CNS layer on top of expansive soil it was intended to produce an environment similar to that existing at no volume change depth in expansive soil.

- CHEMICAL STABILISATION

The chemicals both organic and inorganic can be used to modify the properties of expansive soils.

- CEMENT STABILISATION

Cement stabilisation develops strength due to the cementitious linkage between the calcium silicate and aluminium hydration products and the soil particles.

- LIME STABILISATION

Lime stabilisation is developed due to the base exchange and cementation process between the clay particles and the lime.

- STABILISATION WITH FLYASH

Flyash is artificial pozzolona. It produces hydrous calcium silicate which is of relatively low stability. This attributes to water tightness as well as to strength.

(Chapter 3)

# EXPERIMENTAL WORKS AND PROCEDURES

# TESTS CARRIED OUT

- ## DIFFERENTIAL FREE SWELL TEST

**Procedure**: Pour 10g of dry sample into 100ml graduated cylinder containing about 40 ml of distilled water. The suspension was stirred repeatedly and then made up to the 100 ml mark with the addition of distilled water. These cylinders were sealed and left undisturbed for the sample to settle. After 24 hrs free swell index was calculated.

The free swell index was calculated as
Free swell index = $[V_d - V_k] / V_k \times 100\%$
where,

$V_d$ = volume of soil specimen read from the graduated cylinder containing distilled water.
$V_k$ = volume of soil specimen read from the graduated cylinder containing kerosene.

- ATTERBERG'S LIMITS

Atterberg's limit were calculated firstly in tap water and then in saline water using the IS methods.

✓ LIQUID LIMIT

Liquid limit is the water content corresponding to the arbitrary limit between liquid and plastic state. It is the minimum water content at which the soil is still in the liquid state, but has a small shearing strength against flowing.

## Procedure to Determine The Liquid Limit Of Soil

i) Place a portion of the paste in the cup of the liquid limit device.

ii) Level the mix so as to have a maximum depth of 1cm.

iii) Draw the grooving tool through the sample along the symmetrical axis of the cup, holding the tool perpendicular to the cup.

iv) After the soil pat has been cut by a proper grooving tool, the handle is rotated at the rate of about 2 revolutions per second and the no. of blows counted, till the two parts of the soil sample come into contact for about 10mm length.

v) Take about 10g of soil near the closed groove and determine its water content

vi)The soil of the cup is transferred to the dish containing the soil paste and mixed thoroughly after adding a little more water. Repeat the test.

APPARATUS USED

## Formula Used :

Liquid Limit percent = $w (n/25)^e$

w= water content corresponding to n number of blows

e = index, assumed 0.1 as number of blows are between 20-30.

## ✓ PLASTIC LIMIT

Plastic limit is the water content corresponding to an arbitrary limit between the plastic and the semi-solid states of consistency of a soil.It is defined as the minimum water content at which a soil will just begin to crumble when rolled into a thread approximately 3 mm in diameter.

## Procedure to determine The Plastic Limit Of Soil

i) Take soil sample and roll it with fingers on a glass plate. The rate of rolling should be between 80 to 90 strokes per minute to form a 3mm dia.

ii) If the dia. of the threads can be reduced to less than 3mm, without any cracks appearing, it means that the water content is more than its plastic limit. Knead the soil to reduce the water content and roll it into a thread again.

iii) Repeat the process of alternate rolling and kneading until the thread crumbles.

iv) Collect and keep the pieces of crumbled soil thread in the container used to determine the moisture content.

## ✓ SHRINKAGE LIMIT

Shrinkage limit is defined as the maximum water content at which a reduction in water content will not cause a decrease in volume of a soil mass. It is the lowest water content at which a soil can still be completely saturated.

i) Prepare a wet sample filled completely till the top of the stainless steel shrinkage dish.

ii)Take wet weight of soil sample and then oven dry it.

iii) Then take then volume of oven dry sample by taking the weight of kerosene displaced by it.

iv)Then take the volume of stainless steel shrinkage dish by completely filling it with kerosene.

Formula used :

Shrinkage limit percent = $[ w1 – (V1-Vd)/Wd ] * 100$

(Chapter 4)

# RESULTS

## 4.1 PHYSICAL PROPERTIES OF THE VIRGIN SOIL VIS-A-VIS TAP WATER & SALINE WATER

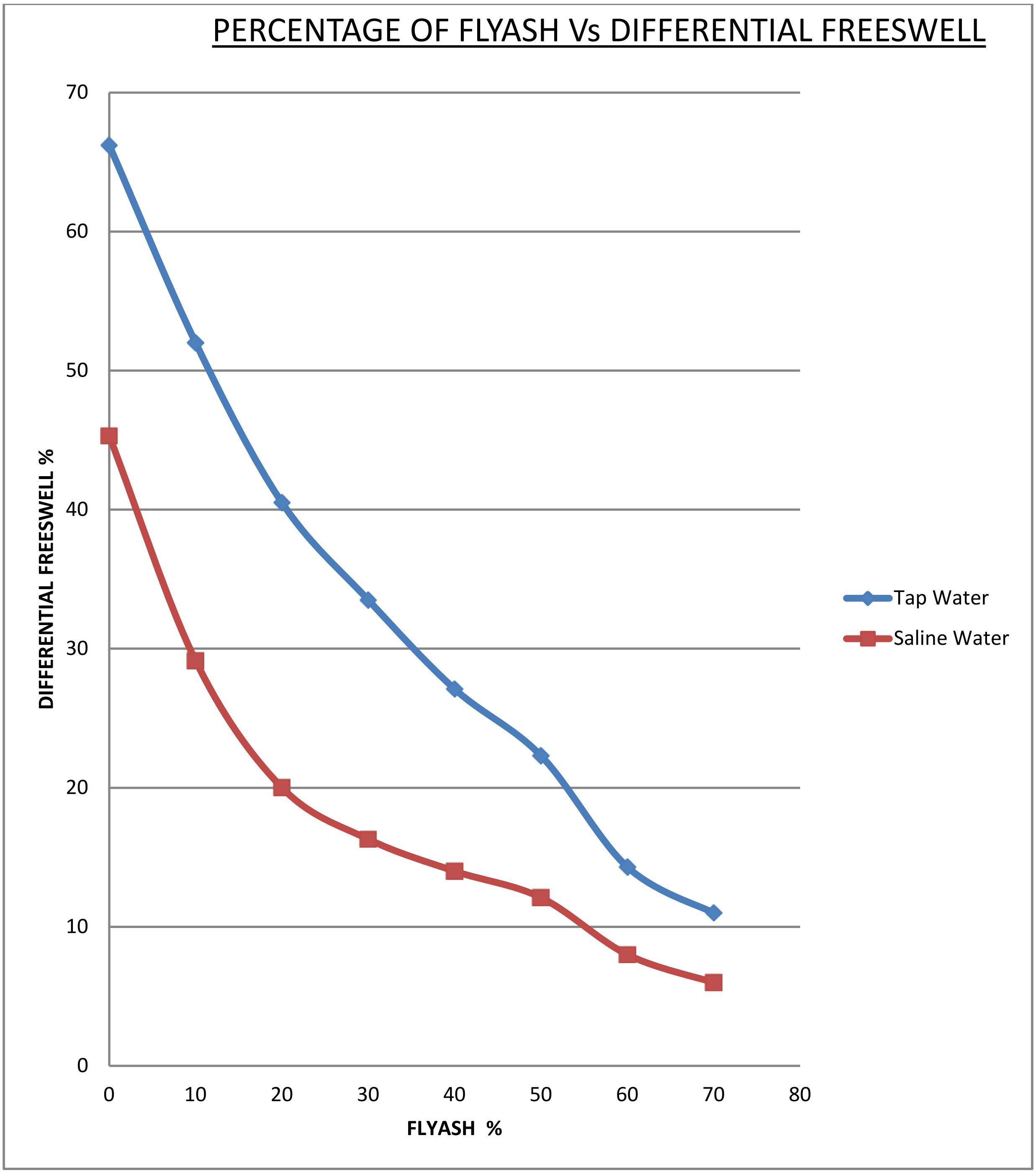
| S. No. | PROPERTIES | TAP WATER | SALINE WATER |
|---|---|---|---|
| 1. | Liquid limit percent | 51.8 | 52.1 |
| 2. | Plastic limit percent | 20.7 | 21.2 |
| 3. | Plasticity index percent | 33.4 | 33.1 |
| 4. | Shrinkage limit percent | 7.24 | 14.4 |
| 5. | Degree of shrinkage percentage | 53.0 | 45.4 |
| 6. | Differential free swell percent | 66.2 | 45.3 |

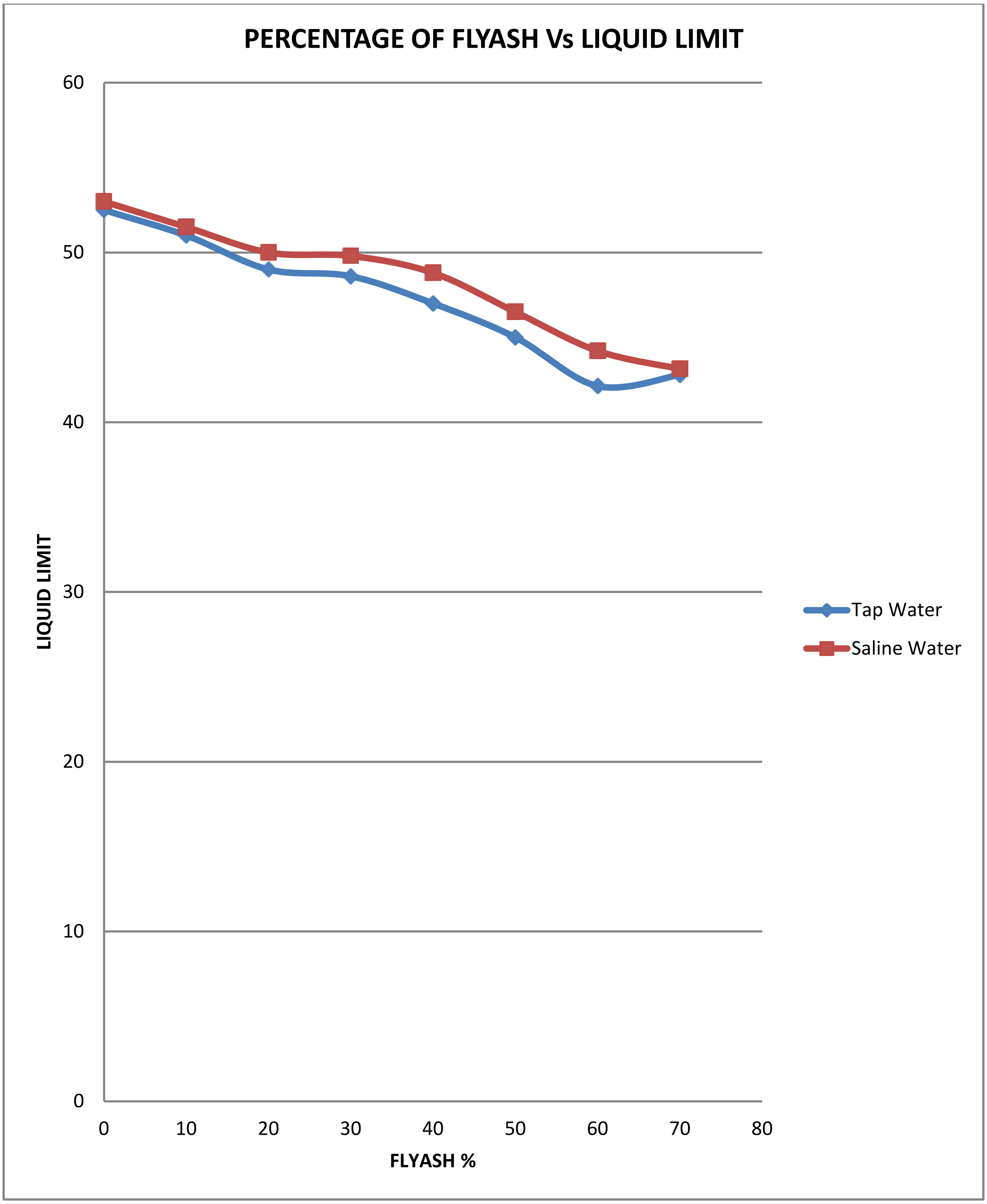
# 4.2 ATTENBERG’S LIMITS & DIFFERENTIAL FREE-SWELL OF SOIL-FLY ASH MIX VIS-A-VIS TAP WATER & SALINE WATER

| S.No. | % of fly ash | Liquid limit percent | | Plastic limit percent | | Shrinkage limit percent | | Differential free swell percent | |
|---|---|---|---|---|---|---|---|---|---|
| | | Tap water | Saline water | Tap water | Saline water | Tap water | Saline water | Tap water | Saline water |
| 1 | 0 | 51.82 | 52.15 | 20.71 | 21.20 | 7.24 | 14.41 | 66.21 | 45.31 |
| 2 | 10 | 51.04 | 51.3 | 21.60 | 22.02 | 10.21 | 15.30 | 53.31 | 30.13 |
| 3 | 20 | 49.13 | 50.2 | 22.81 | 23.01 | 13.32 | 18.30 | 41.01 | 22.32 |
| 4 | 30 | 48.62 | 49.9 | 25.40 | 26.22 | 16.11 | 21.01 | 34.21 | 16.61 |
| 5 | 40 | 47.21 | 48.7 | 31.81 | 32.16 | 19.22 | 24.32 | 28.31 | 15.12 |
| 6 | 50 | 45.21 | 46.6 | 32.91 | 33.21 | 21.41 | 26.31 | 22.50 | 12.01 |
| 7 | 60 | 43.40 | 44.6 | 34.40 | 35.12 | 28.19 | 30.33 | 13.81 | 8.41 |
| 8 | 70 | 42.8 | 43.4 | 35.9 | 36.61 | 30.34 | 32.2 | 12.31 | 7.11 |

(Chapter 5)

# GRAPHS


PERCENTAGE OF FLYASH Vs DIFFERENTIAL FREESWELL
DIFFERENTIAL FREESWELL %
FLYASH %
0
10
20
30
40
50
60
70
80
Tap Water
Saline Water



PERCENTAGE OF FLYASH Vs LIQUID LIMIT
60
50
40
30
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10
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LIQUID LIMIT
0
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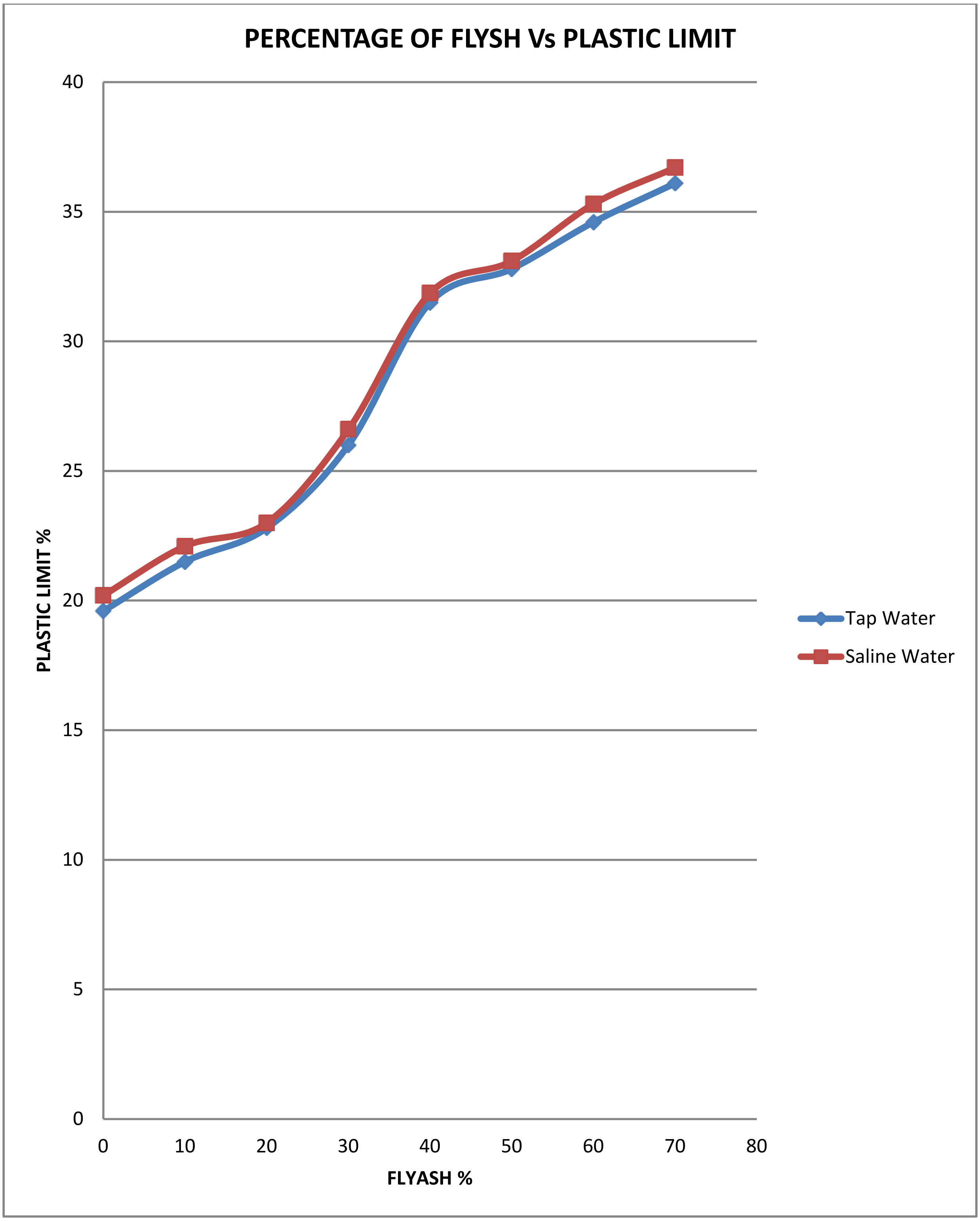
20
30
40
50
60
70
80
FLYASH %
Tap Water
Saline Water


PERCENTAGE OF FLYSH Vs PLASTIC LIMIT
PLASTIC LIMIT %
FLYASH %
40
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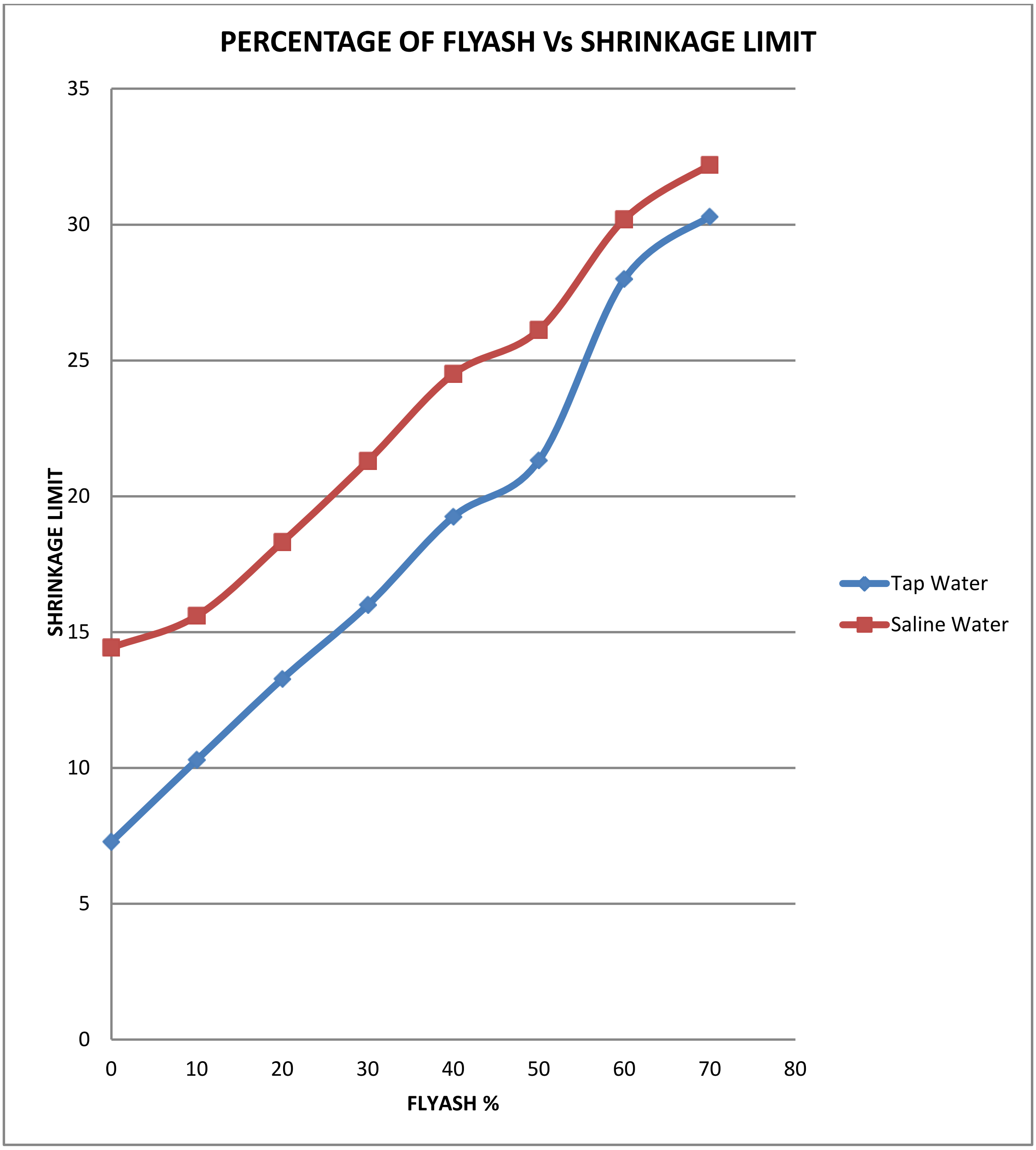
Tap Water
Saline Water


PERCENTAGE OF FLYASH Vs SHRINKAGE LIMIT
35
30
25
20
15
10
5
0
SHRINKAGE LIMIT
0
10
20
30
40
50
60
70
80
FLYASH %
Tap Water
Saline Water

(Chapter 6)

# DISCUSSION

It is an everyday experience that coastal roads are very often inundated whenever high tides occur. Also rise of brackish ground water, mixed with infiltrated sea water contribute to swelling of ground soil near sea shores.

Since saline or brackish water is known to have reduced the effectiveness of lime as a stabilising agent, flyash has been chosen as stabiliser for this investigation. The utilisation of flyash is also helping environment to checking pollution.

Various experiments were conducted by mixing different percentage of flyash to the swelling in the presence of tap water and saline water respectively.

- DIFFERENTIAL FREE SWELL

1) The differential free swell percent decreases with increasing flyash content in presence of both tap water and saline water.

2) In both tap water and saline water the rate of decrease in differential free swell percent is rapid up to 40% of flyash content, thereafter the decrease rate slowed down.

3) The differential free swell percent in saline water is lower than that in tap water, indicating reduction in swelling potential in saline water.

4) The difference in free swell percent between tap water and saline water is remarkable between 0% to 30% of flyash content in the soil. After 30% of flyash content the difference goes on decreasing with increasing flyash content. This shows that after 30% of flyash content in soil, the saline water may not be advantageous a lot over tap water.

- ATTERBERG'S LIMITS

Atterberg's limits, namely liquid limit, plastic limit, shrinkage limit test were conducted on virgin soil and soil- flyash mix in tap water and saline water separately. From the graphs it has been observed that

1) The liquid limit decreases with increasing flyash content.

2) The plastic and shrinkage limit increase with increasing flyash content for both types of water.

3) Values of all Atterberg's limits in saline water are higher than those in tap water. The difference in shrinkage limit values of swelling soil in tap water and sea water is more pronounced than in case of liquid and plastic limit.

4) The plasticity index of soil- flyash mix in saline water is less than that in tap water.

The slope of gain of plastic and shrinkage limit as well as that of decline in liquid limit are steeper between 0 to 30% flyash content than that beyond it. As shrinkage limit is inversely proportional to the degree of shrinkage, steepest gain of shrinkage limit up to 30% flyash content( in both tap and saline water) becomes a significant dividing point.

(Chapter 7)

# CONCLUSION

From the observations of the present investigation and the analysis of the test results, the following conclusions can be drawn:-

1. (a) Differential free swell values of virgin swelling soil in saline water are lesser than those in tap water.

(b) DFS values decreases with increasing flyash content, both in tap water and saline water.

(c) The rate of decrease of differential free swell is remarkable between 0% and 40% of flyash content in both type of water.

2. (a) With the increase of flyash content, liquid limit and plasticity index decrease where the plastic limit and shrinkage limit increases in presence of both tap water and saline water.

(b) Values of Atterberg's limits in saline water are higher than those in tap water.

(c) Degree of shrinkage of virgin swelling soil and swelling soil-flyash mix is less in saline water than in tap water.

In consideration of 1(a) and 2(c) swelling of virgin or flyash mixed swelling soil will be less in saline water. Hence saline water itself acts as a stabilising agent.

(Chapter 8)

# BIBLIOGRAPHY

1) “Swelling Soil- Problems And Remedial Measures”, Central Soil & Materials Research Station, New Delhi.

2)”Katti, R.K. “First IGS Annual Lecture Search For Solutions To Problems In Black Cotton Soils”, Indian Geotechnical Journal Vol-1 No1 PP-1-82

3) IS-1498-1970 “Classification and Identification of Soil for General Engineering Purposes”, Indian Standard Institution, New Delhi.

4) K.R. Arora – “Soil Mechanics & Engineering Foundation”

5) B.M. Das – “Advanced Soil Mechanics” McGraw Hill

6) B.C. Punmia- “Soil Mechanics and Engineering Foundation”

7) IS:2720 (part V)-1972- Atterberg’s limit analysis

8) “Building of Structures on Swelling Soils” The Indian National Society of Soil Mechanics & Foundation Engineering, July -1970

9) www.engineeringcivil.com