On Reducing Bumps at Pavement-Bridge Interface

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YSU Center for Transportation and Materials Engineering (CTME)

Youngstown State University
One University Plaza
Youngstown, OH 44555

Submitted by:

Principal Investigator

AKM Anwarul Islam, PhD, PE
Associate Professor
Department of Civil/Environmental & Chemical Engineering
Youngstown State University
One University Plaza
Youngstown, OH 44555
Phone: (330) 941-1740
Email: aaislam@ysu.edu

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ABSTRACT

This report contains the causes and long-term solutions to the bumps at bridge/approach slab and/or approach slab/pavement interface. A research was conducted on both structural and geotechnical aspects of an approach slab.

A 3-D finite element analysis using ALGOR was performed to find the stresses and deflections for different slab lengths under HL-93 truck loading conditions. Internal moments and applied moments for seven different State Department of Transportation (DOTs) were determined and compared using MathCAD. Laboratory testing was also performed on soil samples for bridges with bumps and without significant bumps around the Youngstown area.

CHAPTER 1

INTRODUCTION

1.1 Definition of the "Bump"

Roadways and embankments are built on sub-grade foundation and compacted fill materials, respectively, that undergo load induced compression over time. The compression leads to settlements. The total settlement of a bridge is usually much smaller than the settlement of the roadway and/or adjoining embankment and results in considerable degree of difference in the settlement at the intersection. Consequently there is a noticeable bump that develops at the bridge ends.

Commonly, the "BUMP" can be defined as the differential settlement at the area between the bridge and approach slab. Stark et al. [1] conducted a survey of 1,181 bridges in the State of Illinois and suggested that the riders' discomfort across the bump was magnified if the approach gradient was in excess of 1/200. Regarding differential settlement at the pavement-bridge interface, Wahls [2] suggested that a differential settlement of 0.5 in. is likely to require maintenance. He also suggested a tolerable relative rotation (differential movement divided by the length over which the settlement occurs) of 1/250 for continuous-span bridges and 1/2000 for simple supported spans.

1.2 Background

Approach slabs are provided at the end of bridges for smooth transition of vehicles. Bridge bumps are a result of differential settlement of bridge/approach slab and/or approach slab/pavement. The most common bumps are at the end of the bridges caused when the soil beneath the approach slab looses contact with it. It can also be caused due to strength deficient approach slabs. This reduces the ride ability of driver and it also brings a bad image of transportation department. Due to repair work often bridges are closed and thus increase the congestion on the roads which causes delay. A vehicle can also lose control due to a bump and it can lead to an accident.

Bridge bumps are a major problem across United States and State Department of Transportation has to spend a huge amount of money as repair cost every year. Briaud et al. [3] summarized that out of 600,000 bridges across United States 150,000 bridges, approximately 25% of total bridges, had bridge bumps that cost approximately $100 million per year for repair. The bridge bumps are the results of unexpected change in the height at the edge of the pavement/approach slab and the slab/bridge deck. To date no multifaceted set of engineering explanations have been developed, since there are many intricate aspects that are involved.

Approach slabs are placed at the end of the bridges for the better transition of a vehicle from a bridge to a pavement. When the soil beneath approach slab looses contact, the slab takes a concave shape. Figure 1.1 shows the behavior of approach slab due to pavement settlement. Approach slab near the bridge is placed on a back wall which is supported on piles (optional). The other end of the approach slab is placed on a sleeper slab (optional). A bump is formed when the backfill soil under the approach slab loses contact with it. It can also be caused due to structural reason if the approach slab is designed insufficiently. Conditions become more severe if the problem is not fixed in a timely manner and with the combination of reoccurring soil settlement and continual impacts from vehicles running over the already created bump. The bridge approach slab deteriorates with daily traffic and becomes progressively bumpier for drivers.

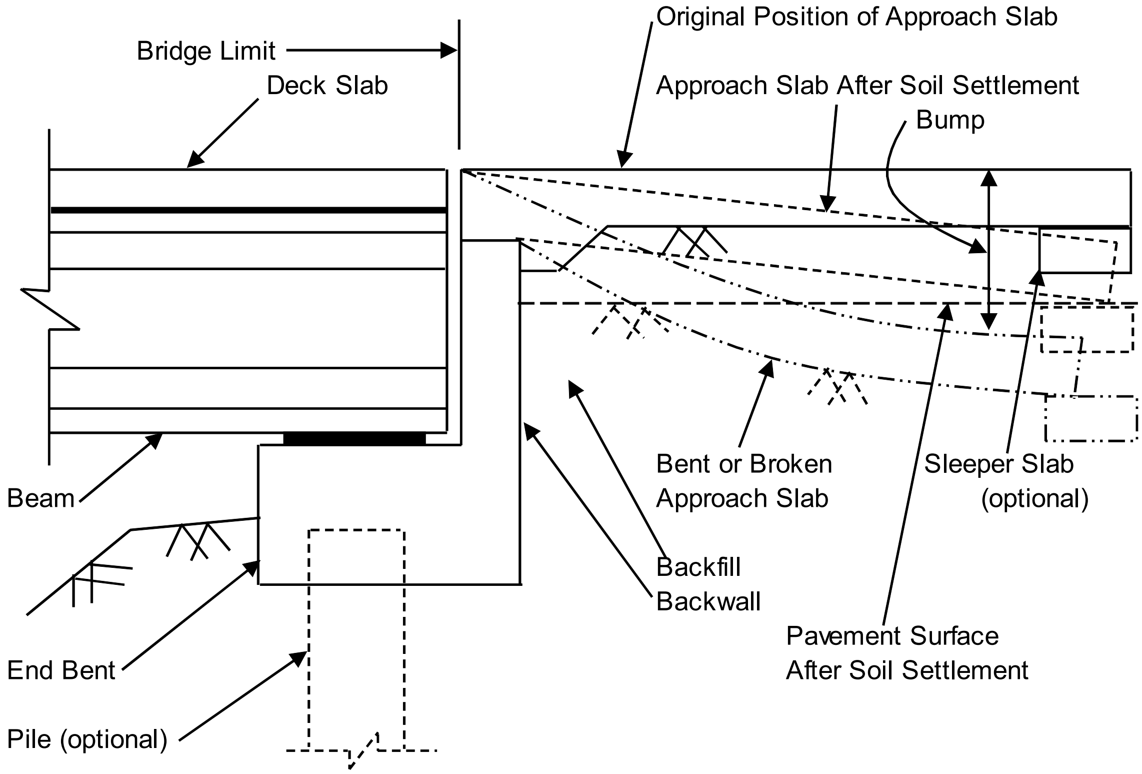

Figure 1.1 – Behavior of Approach Slab due to pavement settlement and bump mechanism

1.3 Goals of Project

The purpose of this research is to find a cost-effective solution to the existing problem of bumps. This will reduce the maintenance cost for the transportation department and it will also provide a smooth ride to drivers.

1.4 Literature Review

A comprehensive literature review was conducted for this research. Cai et al. [4] identified the internal stresses in flat and ribbed approach slabs using a 3-D finite element analysis model. The recommendations were to use ribbed approach slabs for longer spans and to use different sizes of reinforcing beams for different span lengths, both for flat and ribbed approach slabs. Future recommendations for considering both structural and geotechnical research were also explained as settlement is caused by the weight of the rigid slab and the vehicular load.

Dupont and Allen [5] summarized that excessive settlement can occur merely because the design and/or constructions issues were not properly addressed. Issues, such as type of bridge abutment used, joint selection, method of compaction, or basically the approach slab was not constructed according to design are just a few reasons for excessive settlement. The bridge and the abutments are often constructed prior to the final compaction of the approach pavement, creating a difficult situation for compaction equipment to reach near enough to the bridge end. This causes inadequate compaction in the embankment backfill. If the bridge is highly skewed, large compaction equipment cannot operate near the abutment. Insufficient compaction near the abutment is a major contributor to approach distress. Many states must work within a defined budget. Although money may be tight, states are responsible to repair and maintain state highways. When bumps become too high, cost soar which force states to look cheaper and temporary repair solutions. These solutions would include, asphalt wedges, overlays, and milling and filling. Improved permanent solutions are needed to illuminate these bump problems.

According to Stark et al. [1], the differential settlement is considered to be the most predominant cause of approach distress because the settlement of the embankment backfill near the end bent. The difference in elevation at the pavement-bridge interface contributes to bridge bumps that results in increased vehicle damage and a higher pavement maintenance cost. The repair and differential movement is costly and very time consuming. It also causes a possible danger for motorists. Dupont and Allen [5] also noted that the cost of any improved design methods must not exceed the life-cycle maintenance cost of existing practices.

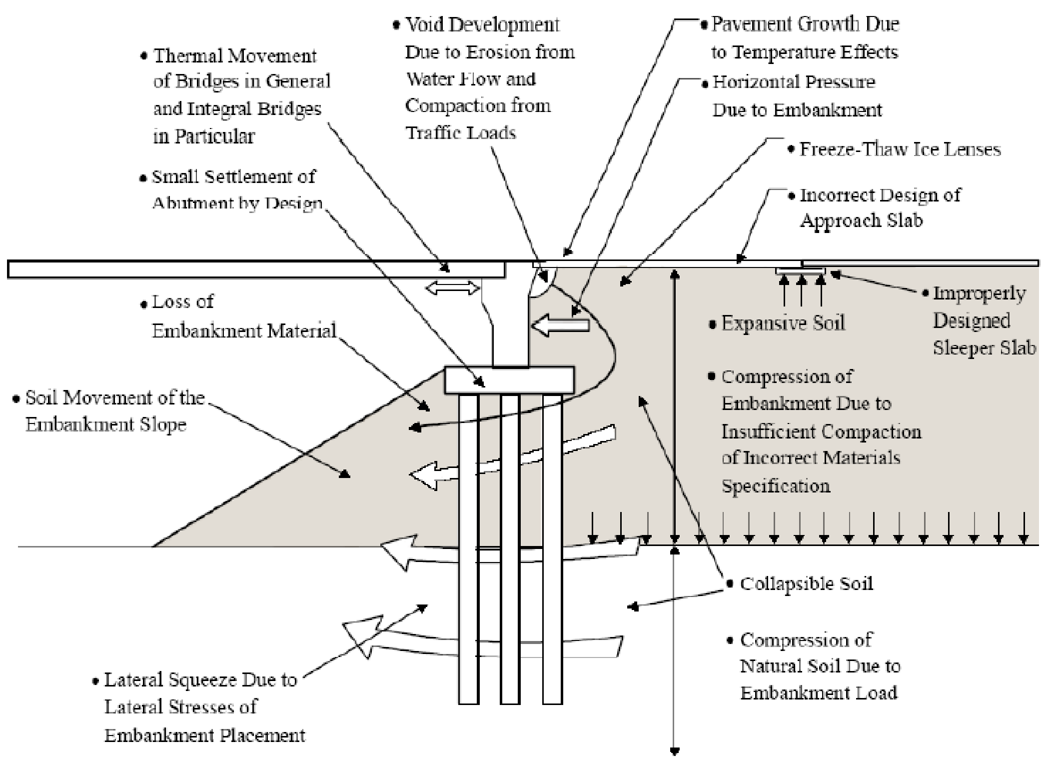

Figure 1.2 - Various parameters that can result in the formation of bumps [3].

An approach slab with International Roughness Index (IRI) of 3.9 mm/m were considered as good riding quality, whereas approach slab with IRI of 10 mm/m or more were poor riding quality [6].

Briaud et al. [3] researched the causes for the settlement of approach slab and listed various reasons which can lead to the formation of bumps shown in Figure 1.2. According to his research, it is important to calculate both the short-term and long-term settlement for the structure. Settlement depends on the type of soil. Rock, gravel and sand deposits show short-term settlement as soon as the load is applied. On the other hand, clay and silts are more likely to have long-term settlement problems. It is advisable to use granular fill materials as they are easy to compact. Foundation type depends on the type of soil. The different types of foundation are pile supported, shallow spread footing, deep spread footing and spread footing on MSE wall. Foundation type also depends if the structure is built over water or not. Settlement also depends on the type of structure and abutment type. The compaction process and the quality of compaction vary depending on the type of abutment. The other factors which affect the performance of bridge approach are bridge-end conditions, construction methods, roadway paving and bridge/roadway joint. Water can seep through poorly maintained joint which results in the erosion of fill material and pressure increases on abutment walls

Foundation soil and embankment play a very significant role in the formation of bumps. Foundation soil should be properly compacted but it is hard to compact the soil at the end of bridges. Thus the soil is loose and within few years of construction of approach slab, it moves out [6].

Puppala et al. [7] surveyed 25 districts in Texas and conducted a comprehensive literature review on previous research and summarized following causes for the settlement of approach slab:

1. Consolidation settlement of foundation soil.
2. Poor compaction and consolidation of backfill material.
3. Poor drainage and soil erosion.
4. Types of bridge abutments.
5. Traffic volume.
6. Age of the approach slab.
7. Approach slab design.
8. Skewness of the bridge.
9. Seasonal temperature variations.

Settlement of soil embankment is one of the major causes of bridge bumps. Hopkins [8] summarized this settlement into three categories:

1. Initial Consolidation

   The initial settlement is the short-term deformation of the foundation when a load is applied to a soil mass. This settlement does not contribute to the formation of bumps, because it occurs prior to the construction of the approach structure [8]. The soil saturation level affects the total contribution of this settlement. For partially saturated soils, the initial settlement will be usually larger than that of saturated soils.

2. Primary consolidation

   Primary settlement is one the main factor that contributes to the total settlement of soils. It occurs over a period of few months for granular soil to few years for clayey soil. Water escapes from granular soil early as compared to clayey soil. The gradual escape of water due to compression of loaded soil is thought to be the reason for this type of settlement. The primary settlement lasts from a few months for granular soils, to a period of up to ten years for some types of clay [9]. The important difference is credited to the larger void ratio and high permeability of granular soils.

3. Secondary Consolidation

   This phase takes place as a result of changes in void ratio of the loaded soil after dissipation of excess pore pressure [8]. In this case, particles and water in the soil accumulation readjust in a synthetic way under a constant applied stress. For the very soft, highly artificial or organic clays, secondary consolidation can be as substantial as the primary consolidation, while in granular soils, it is negligible [9]. It is difficult to compact clayey soil to its optimum moisture and density as compared to sandy soil [9].

To alleviate the settlement, a main objective of any bridge construction project should include a complete investigation of the foundation soil prior to the construction of the approach embankment starts [2]. Previous studies have revealed that the stresses applied to the foundation sub-grades come first and foremost from the embankment loading rather than the bridge or traffic loads, apart from the shallow depths (less than 10 ft). Geotechnical studies must be conducted with extensive foundation investigations, together with laboratory test to assess the compressions and consolidation potential to better estimate the predictable post-construction settlements [5]. It is also important to study the potential shear failures in the foundation that causes lateral deformations and exterior settlement problems. This kind of failure is more probable to appear in peat and organic materials.

Seasonal changes in the air temperature can also have an effect on bridges and superstructures. The expansion and contraction due to warm and cold temperatures can cause a cyclic loading that is subjected towards the approach backfill and the foundation. High temperatures cause the bridge deck to expand and the bridge abutment also moves against the retained embankment soil. The side movement generates the stress in the soil, causing sometimes for it to reach the passive limit [10].

When the temperature lowers, the superstructure and the abutment moves away from the soil, leaving a void at the interface between the abutment and the backfill. The voids get larger as the temperature drops. Soil voids can cause erosion; this increases the size of the void behind the abutment and below the approach slab, as shown in Fig. 1.3.

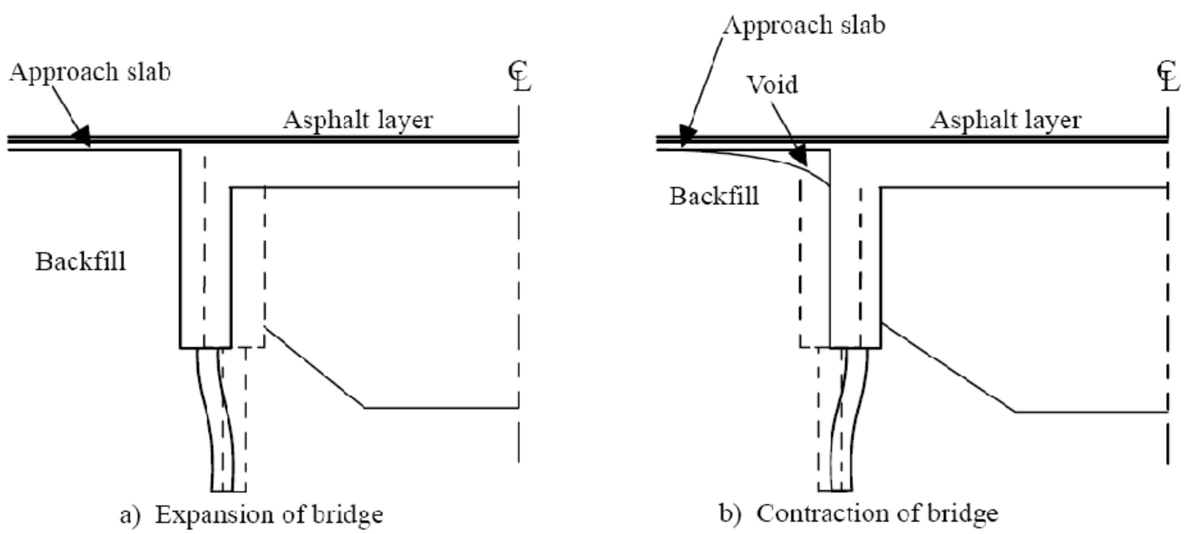

Figure 1.3 – Movement of Bridge Structure with Temperature [12].

According to Wahls [2] bridge abutment can be improved by installing compressible elastic materials between the abutment and the backfill. This material should have elastic properties that permit large recoverable cyclic movement and hydraulic properties that would allow adequate drainage without causing erosion from the backfill.

Dupont and Allen [5] conducted a survey on 50 state highways agencies in the United States. Their study showed that only thirty one (31) states used approach slabs. Of the thirty one states, only fourteen (14) states used the sleeper. The purpose for the design of the sleeper slab was to diminish the possibility of the differential settlement by letting the approach slab settle with the embankment. This prevents the bump at the bridge. If the sleeper slab is designed wrong it will cause settlement problems [12]. When the expansion joints are placed on the top of the sleeper slab, there is a possibility of cracking and crushing of the approach slab concrete due to the expansion joints and dragging of the approach slab [13,14]. Seo et al. [14] proposed that the width of the sleeper slab should be 5 ft, as shown Fig. 1.4

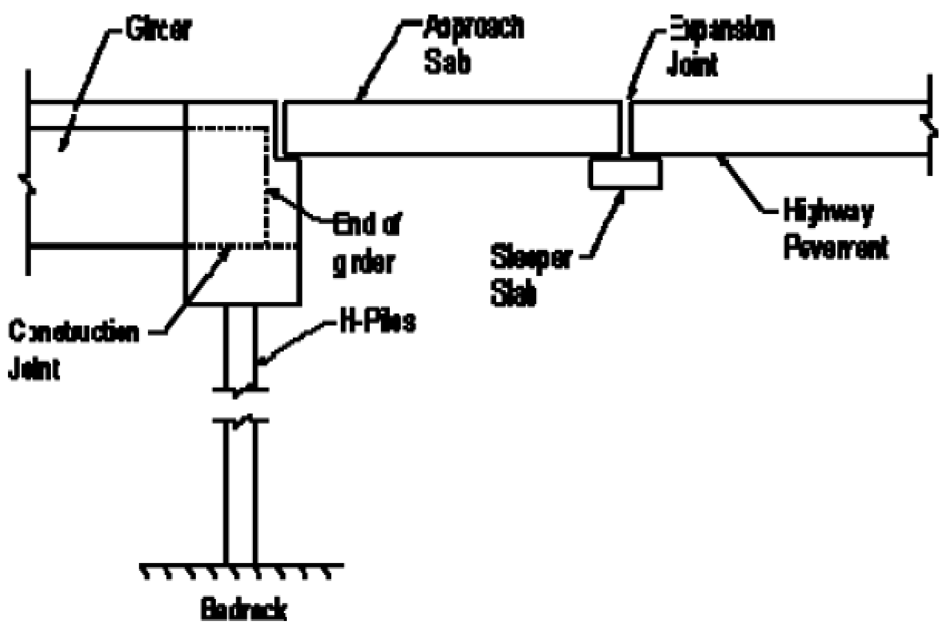

Figure 1.4 – Schematic of an Integral Abutment System with Sleeper Slab [14].

To eliminate design flaws with the sleeper slabs, the first system consists with placing higher quality of MSE backfill, also known as backfill, under the sleeper slab instead of under the approach slab. The second sustaining system is made of driven piles to support the sleeper slab. Cheaper materials are used for backfill behind the abutments and expansion joint device that is placed on top of the sleeper slab [13].

Ineffective drainage and erosion control methods are additional problems often are credited to the settlements near bridge abutments. Underlying fill materials that are allowed to become wet from water collected on the bridge pavement due to ineffective seals, can flow and erode to underlying backfill materials. This erosion can cause a void under the bridge abutments causing the eventual settlements of the bridge approach slabs. The design of the bridge approaches must be incorporated with a well-organized drainage system. This could also include drainage inlets at the end of the bridge deck so surface water could be redirected before it reaches the approach slab [13].

Surface or internal drainage that keeps water off the slopes is suggested for correcting the superficial erosion of embankments [2]. Keeping water away from the soil is very important in plummeting the settlement of the soil. A good drainage system should be incorporated with the construction costs. The drainage system’s cost is low when compared to the maintenance costs that will be needed throughout the life of the bridge if the drainage is poor [5].

The pressure of voids under the approach slab can lead to cracking, sinking, instability, and pounding problems [13]. This method is used for bridge approach maintenance, as a practice of preventive measure [6]. Pressure grouting is used to fill voids under the approach slab with the injection of flow able grout, without having to raise the slab [13].

1.5 Research Significance

The goal of this research is to establish a more cost efficient solution for reducing bumps at pavement- bridge interface to improve ride excellence that will decrease safety hazards and maintenance costs. In search of this objective, the research aim will be to develop more advanced innovations and guidelines for the design and construction of approach slab adjoining embankment that will minimize or prevent bridge bumps to an acceptable level.

Replacement method for the deteriorated approach slabs due to the formation of a bump are the most expensive and time consuming as the construction process leads to infrequent closure of lanes and traffic congestion.

CHAPTER 2

EXPERIMENTAL INVESTIGATIONS

As discussed in earlier chapters, soil is one of most important factor which should be considered for the differential settlement of pavement/bridge interface. The total load on the slab which is dead load and live load is all finally transferred to the soil. A soil should be strong enough to bear all the loads otherwise the structure would fail.

Weathering of rocks results in the formation of soil. Weathering can occur in either of the two ways, mechanical and chemical. The kind of soil produced depends on the kind of rock which results in its formation. Soils are classified by their particle size, namely gravel, sand, silt and clay. They have different properties and behave differently when a load is applied to them. Table 2.1 shows particle-size classification developed by the U.S. Department of Agriculture, the Massachusetts Institute of Technology, the American Association of State Highway and Transportation Officials, and the U.S. Army Corps of Engineers and U.S. Bureau of Reclamation [15].

Several bridges with significant bumps and without bumps in Columbiana County were visited and soil samples were collected. Soil collected from these sites was then brought in the laboratory and were tested. The various tests performed on soil were Sieve Analysis, Liquid Limit and Plastic Limit. These tests were performed in order to classify soil according to American Association of State Highway and Transportation Officials (AASHTO) [16] Classification System. Table 2.2 shows the U.S. standard sieve numbers and the sizes of openings [15].

Table 2.1 – Particle-size Classifications

Name of Organization	Grain size (mm)			
	Gravel	Sand	Silt	Clay
Massachusetts Institute of Technology (MIT)	>2	2 to 0.06	0.06 to 0.002	<0.002
U.S. Department of Agriculture (USDA)	>2	2 to 0.05	0.05 to 0.002	<0.002
American Association of State Highway and Transportation Officials (AASHTO)	76.2 to 2	2 to 0.075	0.075 to 0.002	<0.002
Unified Soil Classification System (U.S. Army Corps of Engineers, U.S. Bureau of Reclamation, and American Society for Testing and Materials	76.2 to 4.75	4.75 to 0.075	Fines (i.e. silts and clays) <0.075	

Note: Sieve openings of 4.75 mm are found on a U.S. No. 4 sieve; 2 mm openings on a U.S. No. 10 sieve; 0.075 mm openings on a U.S. No. 200 sieve. See Table 3.2.

Table 2.2 – U.S. Standard Sieve Sizes

Sieve no.	Opening (mm)	Sieve no.	Opening (mm)
4	4.75	35	0.500
5	4.00	40	0.425
6	3.35	50	0.355
7	2.80	60	0.250
8	2.36	70	0.212
10	2.00	80	0.180
12	1.70	100	0.150
14	1.40	120	0.125
16	1.18	140	0.106
18	1.00	170	0.090
20	0.850	200	0.075
25	0.710	270	0.053
30	0.600		

The U.S. Army Corps of Engineers (1949) at the Waterways Experiment Station in Vicksburg, Mississippi, proposed an empirical equation for the calculation of Liquid Limit. All the calculations were performed using this formula which is given below:

$$LL = W\,(N/25)^{0.12}$$

where, N = number of drops of the cup required to close the groove at the moisture content, W. Table 2.3 shows values of $(N/25)^{0.12}$ for different values of N taken from Geotechnical Test Method (GTM-7 Revision #1), Geotechnical Engineering Bureau, New York State Department of Transportation (April 2007).

Table 2.3 – Values of $(N/25)^{0.12}$

N	**$(N/25)^{0.12}$**	**N**	**$(N/25)^{0.12}$**
15	0.941	23	0.990
16	0.948	24	0.995
17	0.955	25	1.000
18	0.961	26	1.005
19	0.967	27	1.009
20	0.974	28	1.014
21	0.979	29	1.018
22	0.985	30	1.022

Bridges with significant bumps and bridges without bumps were visited in Columbiana County and soil samples were collected. Table 2.4 shows the results which were obtained by the laboratory experiments on these soil samples.

Table 2.4 – Results obtained on different soil samples

Results on	**Bridges without bumps**		**Bridges with bumps**		
	COL 30 2578	COL 30 2667	COL 30 11 2L	COL 30 2670	COL 30 3182
Liquid Limit	34.1	21.5	24.9	33.6	32.2
Plastic Limit	24.5	25.5	21.9	30.7	38.4
Plasticity Index	9.6	NP	3.4	2.9	NP
Soil Classification	A-2-4 (0)	A-3 (0)	A-1-b (0)	A-1-b (0)	A-3 (0)

Note: Soil samples were collected from the surface i.e. top layer

The results obtained after performing various experiments on collected soil show that soil from all different locations are granular. After few years of construction of pavement it is usually seen that the soil close to the end bent moves out. The most important factor for this problem is the improper compaction of soil. Compaction is removal of air from the soil and thus makes it denser. Water is added to the soil and various instruments are used to compact it properly. The quantity of water that would be added to the soil should be close to the optimum moisture content. Soil type and moisture content affect the quality of compaction. Proper compaction of the granular soil can be attained by Pneumatic rubber-tired rollers, Vibratory rollers and Hand-held vibrating plates. One of the most famous compaction techniques in United States is Dynamic compaction. Either of these compaction techniques could be used depending on the field conditions.

Calculations for Soil Classification of COL 30 2578

Sieve Analysis Test

Table 2.5 – Sieve Analysis Test

Sieve Size	Sieve Wt (g)	Sieve+Sample Wt (g)	Mass Retained (g)	% on each sieve	Cumulative	Sieve dia(mm)	%Finer
1"	830	830	0	0.000	0.000	25.400	100.000
3/4"	830	830	0	0.000	0.000	19.050	100.000
3/8"	542.6	559.9	17.3	3.327	3.327	9.525	96.673
4	514.5	626	111.5	21.442	24.769	4.750	75.231
10	484.6	653.2	168.6	32.423	57.192	2.000	42.808
40	378.7	563.4	184.7	35.519	92.712	0.425	7.288
100	328	353.3	25.3	4.865	97.577	0.150	2.423
200	325	331.1	6.1	1.173	98.750	0.075	1.250
Pan	500	506.2	6.2	1.192	99.942		0.058

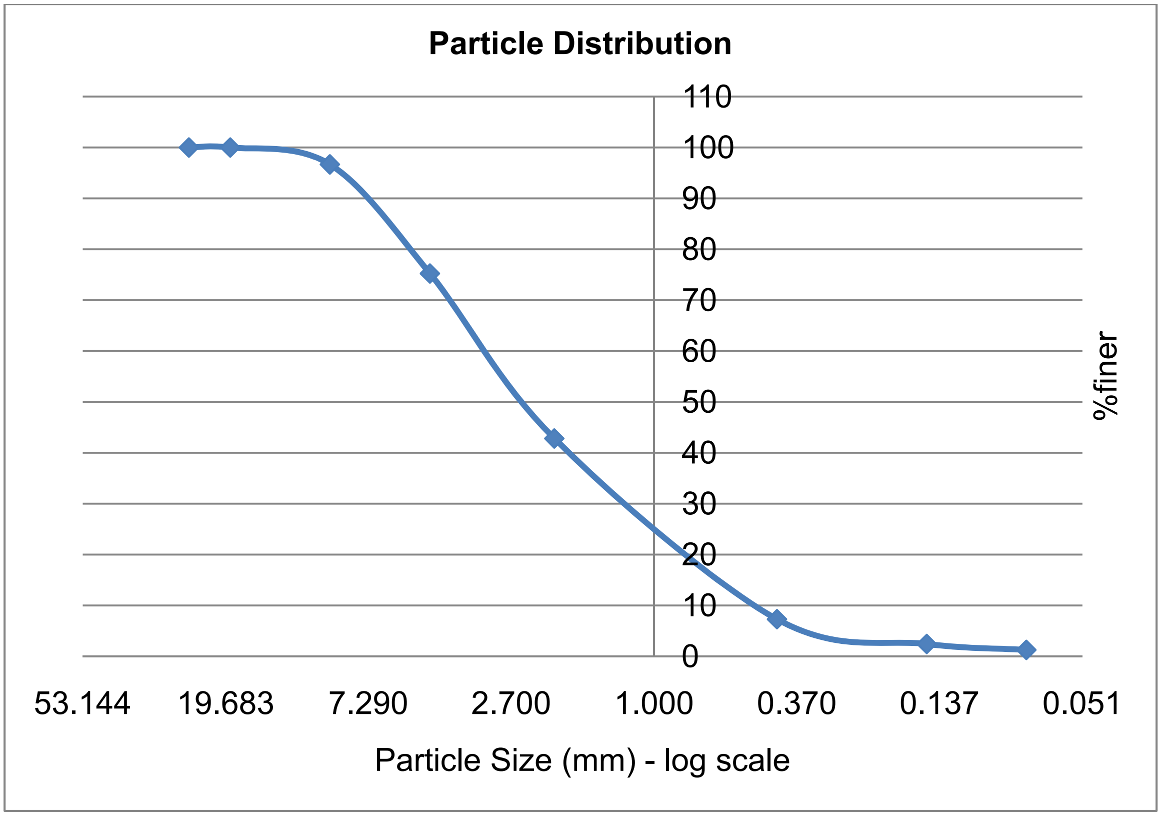

Figure 2.1 – Particle-size Distribution

Liquid Limit Test

Table 2.6 – Liquid Limit test

Number of drops	24
Mass of can + moist soil (g)	223
Mass of can + dry soil (g)	204.4
Mass of can (g)	150.1
Mass of water (g)	18.6
Mass of dry soil (g)	54.3
Moisture content (%)	34.3

Moisture content (%), W = (Mass of water/Mass of dry soil) * 100

Liquid Limit, LL = W $(N/25)^{012}$

= 34.3 * 0.995

LL= 34.1

Plastic Limit Test

Table 2.7 – Plastic Limit Test

	Sample 1	**Sample 2**
Mass of can + moist soil (g)	41.3	42
Mass of can + dry soil (g)	39.5	40
Mass of can (g)	32.1	31.9
Mass of water (g)	1.8	2
Mass of dry soil (g)	7.4	8.1
Water content (%)	24.32432	24.69136
Plastic limit (%)	24.5	

Plasticity Index (PI) = Liquid Limit – Plastic Limit

= 34.1 – 24.5

= 9.6

Group Index (GI) = (F_{200} – 35) [0.2 + 0.005 (LL – 40)] + 0.01 (F_{200} – 15) (PI – 10)

where, F_{200} = percentage passing through the No. 200 sieve

LL = Liquid Limit

PI = Plasticity Index

Since Liquid Limit of is less than 40 and Plasticity Index is less than 10. Therefore, Group index will be negative and thus GI is taken as 0.

Group Index (GI) = 0

The classification for soil is **A-2-4 (0)**.

Calculations for Classification of COL 30 2667

Sieve Analysis Test

Table 2.8 – Sieve Analysis Test

Sieve Size	Sieve Wt (g)	Sieve+Sample Wt (g)	Mass Retained (g)	% on each sieve	Cumulative	Sieve dia(mm)	%Finer
1"	830	830	0	0.000	0.000	25.400	100.000
3/4"	830	830	0	0.000	0.000	19.050	100.000
3/8"	531.7	543.8	12.1	2.327	2.327	9.525	97.673
4	505.9	530.9	25	4.808	7.135	4.750	92.865
10	484.6	576.9	92.3	17.750	24.885	2.000	75.115
40	378.7	542.5	163.8	31.500	56.385	0.425	43.615
100	328	452.6	124.6	23.962	80.346	0.150	19.654
200	325	356	31	5.962	86.308	0.075	13.692
Pan	499.6	527.2	27.6	5.308	91.615		8.385

Note: For A-3 the %finer than 40 sieve should at least be 51 but sieve analysis gives a value of 43.615 for %finer than 40 sieve. The error in the results can be anticipated due to the fact that soil collected was not a sub-grade soil.

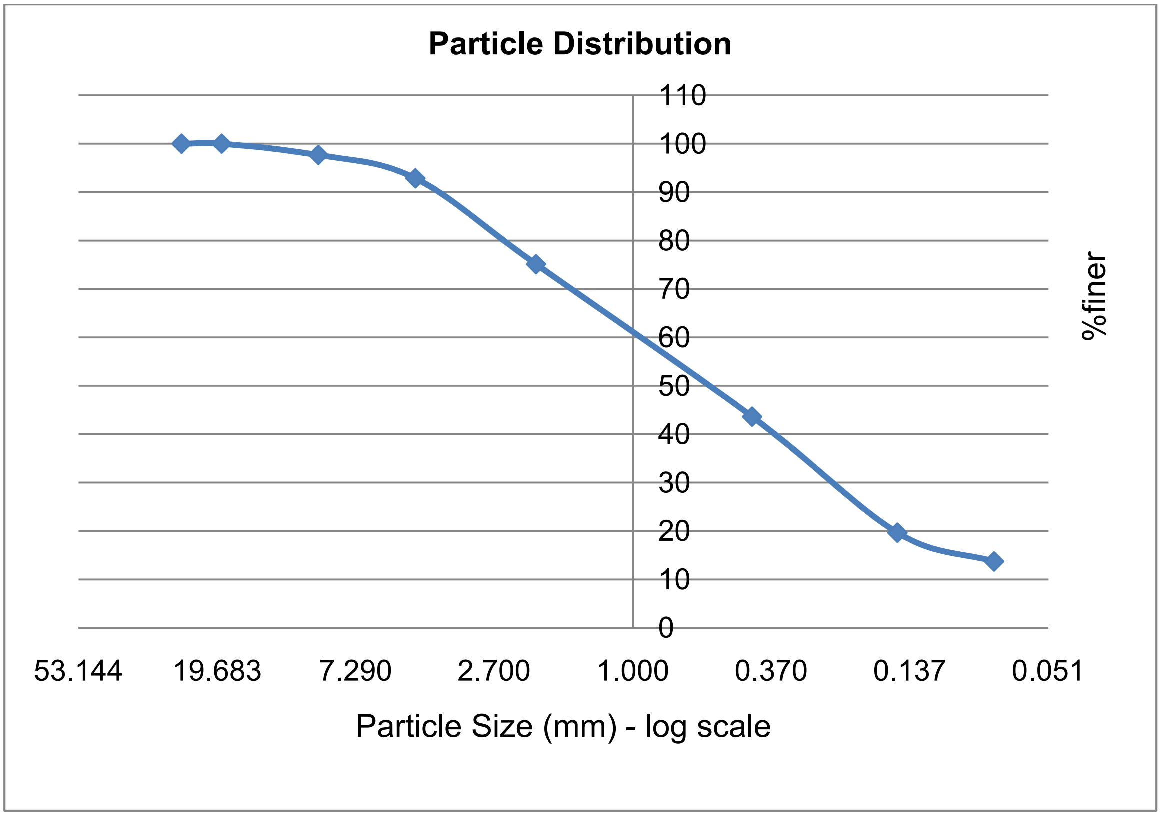

Figure 2.2 – Particle-size Distribution

Liquid Limit Test

Table 2.9 – Liquid Limit Test

Number of drops	23
Mass of can + moist soil (g)	95
Mass of can + dry soil (g)	83.7
Mass of can (g)	31.7
Mass of water (g)	11.3
Mass of dry soil (g)	52
Moisture content (%)	21.7

Liquid Limit, LL = W $(N/25)^{012}$

= 21.7 * 0.990

LL= 21.5

Plastic Limit Test

Table 2.10 – Plastic Limit Test

	Sample 1	**Sample 2**
Mass of can + moist soil (g)	41.5	42.1
Mass of can + dry soil (g)	39.7	40
Mass of can (g)	32.5	31.9
Mass of water (g)	1.8	2.1
Mass of dry soil (g)	7.2	8.1
Water content (%)	25	25.9
Plastic limit (%)	25.5	

Plasticity Index (PI) = Liquid Limit – Plastic Limit

Since LL= 21.5 is smaller than PL= 25.5, therefore PI is negative and hence the soil is non-plastic. Group Index (GI) is also negative and taken as 0. The classification of soil is **A-3 (0)**.

Calculations for Soil Classification of COL 11 2L

Sieve Analysis Test

Table 2.11 – Sieve Analysis Test

Sieve Size	Sieve Wt (g)	Sieve+Sample Wt (g)	Mass Retained (g)	% on each sieve	Cumulative	Sieve dia(mm)	%Finer
1"	830	830	0	0.000	0.000	25.400	100.000
3/4"	830	830	0	0.000	0.000	19.050	100.000
3/8"	531.7	534.8	3.1	0.596	0.596	9.525	99.404
4	505.9	533.1	27.2	5.231	5.827	4.750	94.173
10	484.6	630.8	146.2	28.115	33.942	2.000	66.058
40	378.7	538.7	160	30.769	64.712	0.425	35.288
100	328	431.7	103.7	19.942	84.654	0.150	15.346
200	325	356.8	31.8	6.115	90.769	0.075	9.231
Pan	499.6	526.3	26.7	5.135	95.904		4.096

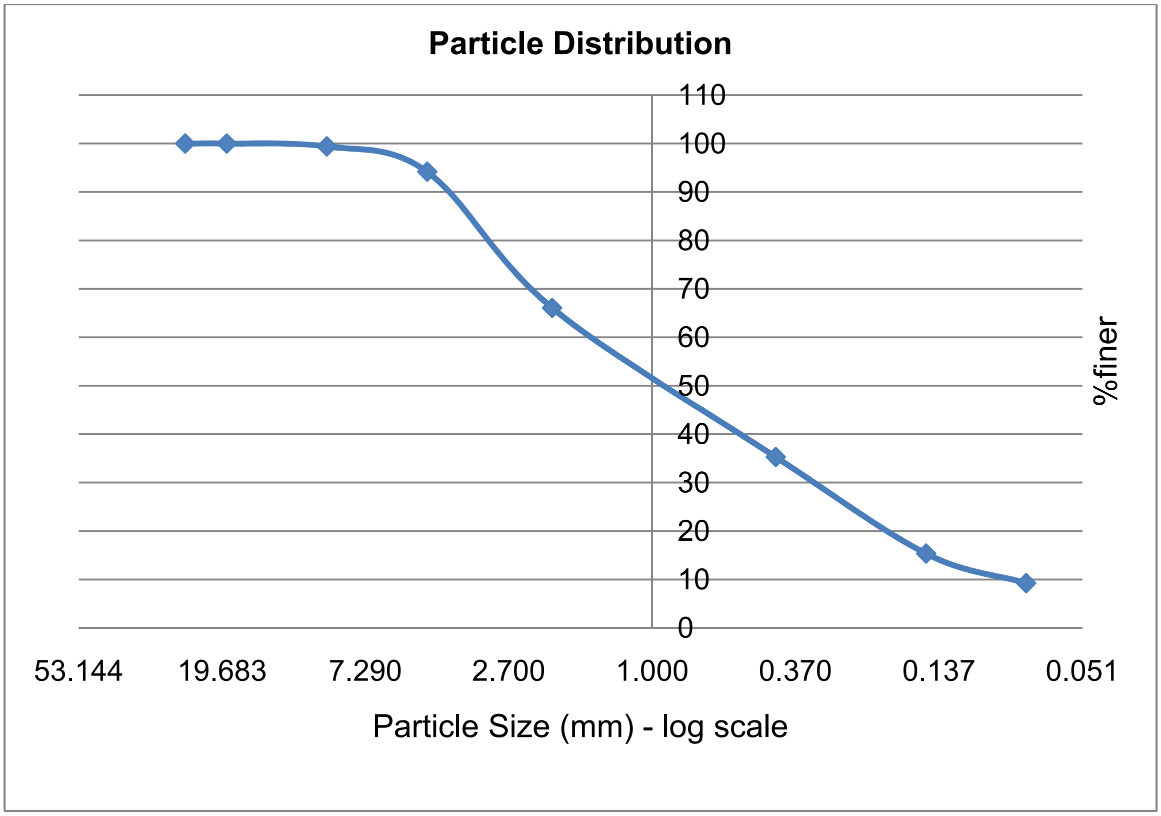

Figure 2.3 – Particle-size Distribution

Liquid Limit Test

Table 2.12 – Liquid Limit Test

Number of drops	16
Mass of can + moist soil (g)	64.9
Mass of can + dry soil (g)	58
Mass of can (g)	31.8
Mass of water (g)	6.9
Mass of dry soil (g)	26.2
Moisture content (%)	26.3

Liquid Limit, LL = W $(N/25)^{012}$

= 26.3 * 0.948

LL= 24.9

Plastic Limit Test

Table 2.13 – Plastic Limit test

	Sample 1	**Sample 2**
Mass of can + moist soil (g)	42.3	43.4
Mass of can + dry soil (g)	40.6	41.4
Mass of can (g)	32	32.8
Mass of water (g)	1.7	2
Mass of dry soil (g)	8.6	8.6
Water content (%)	19.8	23.3
Plastic limit (%)	21.5	

Plasticity Index (PI) = Liquid Limit (LL) – Plastic Limit (PL)

= 24.9 – 21.5

= 3.4

Group Index is also negative since LL is less than 40 and PI is less than 10 and thus GI is taken as 0. The classification of soil is **A-1-b (0)**.

Calculations for classification for soil of COL 30 2670

Sieve Analysis Test

Table 2.14 – Sieve Analysis Test

Sieve Size	Sieve Wt (g)	Sieve+Sample Wt (g)	Mass Retained (g)	% on each sieve	Cumulative	Sieve dia(mm)	%Finer
1"	830	830	0	0.000	0.000	25.400	100.000
3/4"	830	830	0	0.000	0.000	19.050	100.000
3/8"	542.7	549.4	6.7	1.288	1.288	9.525	98.712
4	514.6	586.1	71.5	13.750	15.038	4.750	84.962
10	484.6	665	180.4	34.692	49.731	2.000	50.269
40	378.7	561	182.3	35.058	84.788	0.425	15.212
100	328	373	45	8.654	93.442	0.150	6.558
200	325	333.9	8.9	1.712	95.154	0.075	4.846
Pan	499.6	505.3	5.7	1.096	96.250		3.750

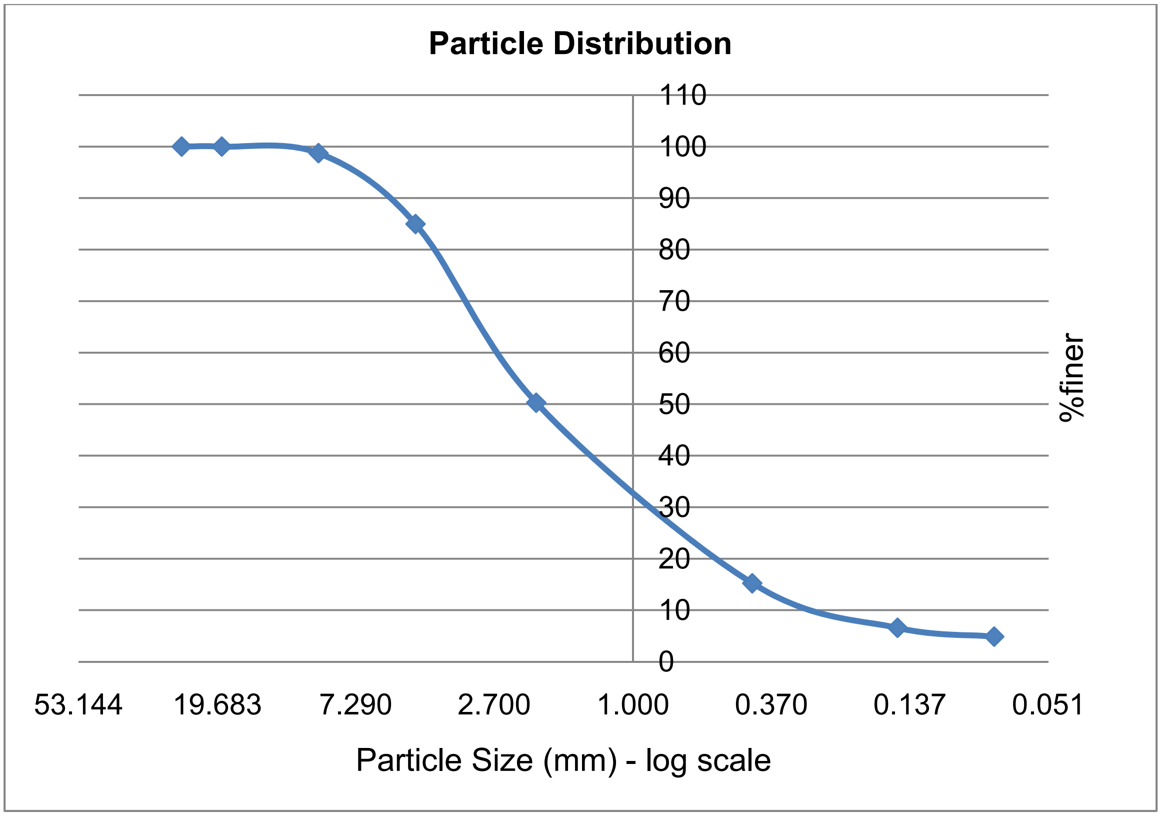

Figure 2.4 – Particle-size Distribution

Liquid Limit Test

Table 2.15 – Liquid Limit Test

Number of drops	18
Mass of can + moist soil (g)	45.5
Mass of can + dry soil (g)	42
Mass of can (g)	32
Mass of water (g)	3.5
Mass of dry soil (g)	10
Moisture content (%)	35

Liquid Limit, LL = W $(N/25)^{012}$

= 35 * 0.961

LL= 33.6

Plastic Limit Test

Table 2.16 – Plastic Limit Test

	Sample 1	**Sample 2**
Mass of can + moist soil (g)	41.1	41
Mass of can + dry soil (g)	39	38.8
Mass of can (g)	32	31.8
Mass of water (g)	2.1	2.2
Mass of dry soil (g)	7	7
Water content (%)	30	31.4
Plastic limit (%)	30.7	

Plasticity Index (PI) = Liquid Limit (LL) – Plastic Limit (PL)

= 33.6 – 30.7

= 2.9

Group Index (GI) is negative and taken as 0. The classification of the soil is **A-1-b (0)**.

Calculations for classification of COL 30 3182

Sieve Analysis

Table 2.17 – Sieve Analysis Test

Sieve Size	Sieve Wt (g)	Sieve+Sample Wt (g)	Mass Retained (g)	% on each sieve	Cumulative	Sieve dia(mm)	%Finer
1"	830	830	0	0.000	0.000	25.400	100.000
3/4"	830	830	0	0.000	0.000	19.050	100.000
3/8"	542.7	552.7	10	1.923	1.923	9.525	98.077
4	514.6	557.7	43.1	8.288	10.212	4.750	89.788
10	484.6	542.7	58.1	11.173	21.385	2.000	78.615
40	378.7	595.6	216.9	41.712	63.096	0.425	36.904
100	328	452.6	124.6	23.962	87.058	0.150	12.942
200	325	357.2	32.2	6.192	93.250	0.075	6.750
Pan	499.6	516.9	17.3	3.327	96.577		3.423

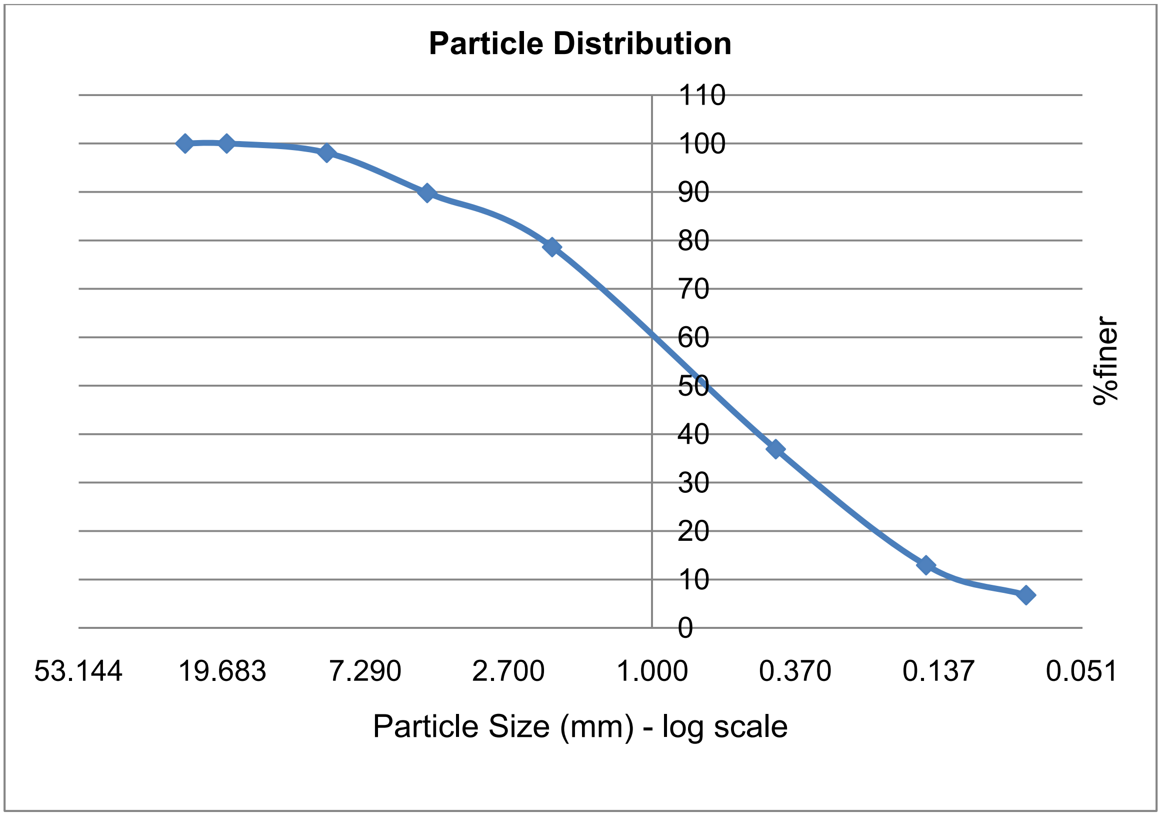

Figure 2.5 – Particle-size Distribution

Liquid Limit Test

Table 2.18 – Liquid Limit Test

Number of drops	28
Mass of can + moist soil (g)	51.6
Mass of can + dry soil (g)	46.8
Mass of can (g)	31.7
Mass of water (g)	4.8
Mass of dry soil (g)	15.1
Moisture content (%)	31.8

Liquid Limit, LL = W $(N/25)^{012}$

= 31.8 * 1.014

LL= 32.2

Plastic Limit Test

Table 2.19 – Plastic Limit Test

	Sample 1	**Sample 2**
Mass of can + moist soil (g)	41.5	43
Mass of can + dry soil (g)	39.1	39.8
Mass of can (g)	32.4	32
Mass of water (g)	2.4	3.2
Mass of dry soil (g)	6.7	7.8
Water content (%)	35.8	41.0
Plastic limit (%)	38.4	

Plasticity Index (PI) is negative since Plastic Limit is greater than Liquid Limit. Therefore, soil is non-plastic. Also, Group index is negative too and taken as 0. The classification of the soil is **A-3 (0)**.

CHAPTER 3

ANALYTICAL SIMULATIONS

This chapter contains the research on the design parameters of an approach slab. Approach slab is placed at the end of the bridges with its one end resting on an end bent and the other on a sleeper slab, which is optional. Movement of soil beneath the abutment is one of the most prominent causes for the formation of bumps. A finite element analysis research was conducted on the design parameters like stresses and deflections by building two separate models in ALGOR. The first model had an approach slab with the soil underneath it while the second model had an approach slab without soil underneath it.

A Linear Stress Analysis and MES Analysis were performed in ALGOR, which is a finite element analysis application. The first model had an approach slab of 30 ft. in length and 20 ft. in width having a thickness of 17 in. with soil underneath it as shown in Fig. 3.1. A sleeper slab and end bent were also built on each of the ends of approach slab and all three structures were considered to be medium strength concrete with boundary conditions as fixed. Approach slab was reinforced with #10 reinforcing steel bars as bottom reinforcement @ 6.5 in c.c. Top reinforcement of #5 bars @ 18 in c.c. was also provided with a bent reinforcement of #5 bars @ 18 in c.c. Dead load of the whole structure was also considered and an HL-93 single lane truck load was placed on the slab in such a way that it produces maximum deflection and moment on the slab. The tire area on the slab was taken as 9 in. in length and 18 in. in width. In the first model the soil properties were user defined having a Poisson’s ratio of 0.25 and

Modulus of Elasticity of 6500 lbf/in^2. The second model is also made with the same characteristics but soil was not considered as shown in Fig. 3.2.

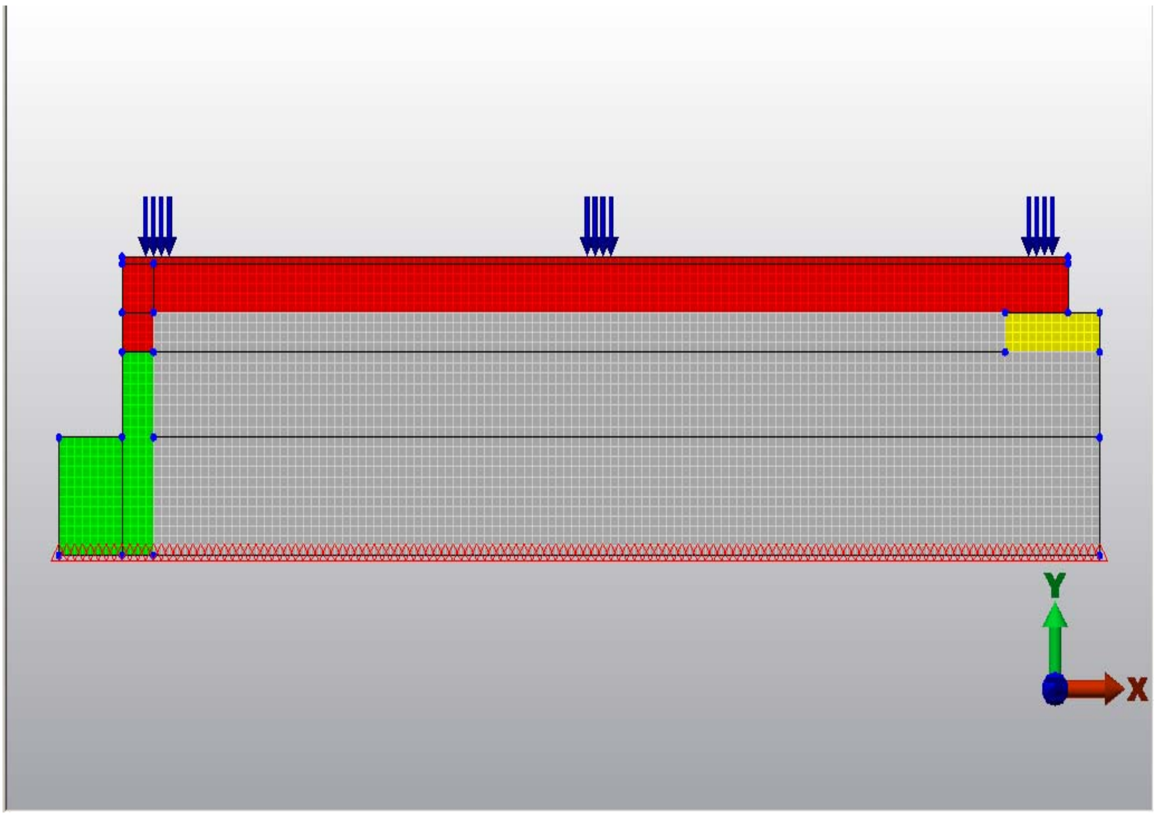

Figure 3.1 – FEM of approach slab with soil.

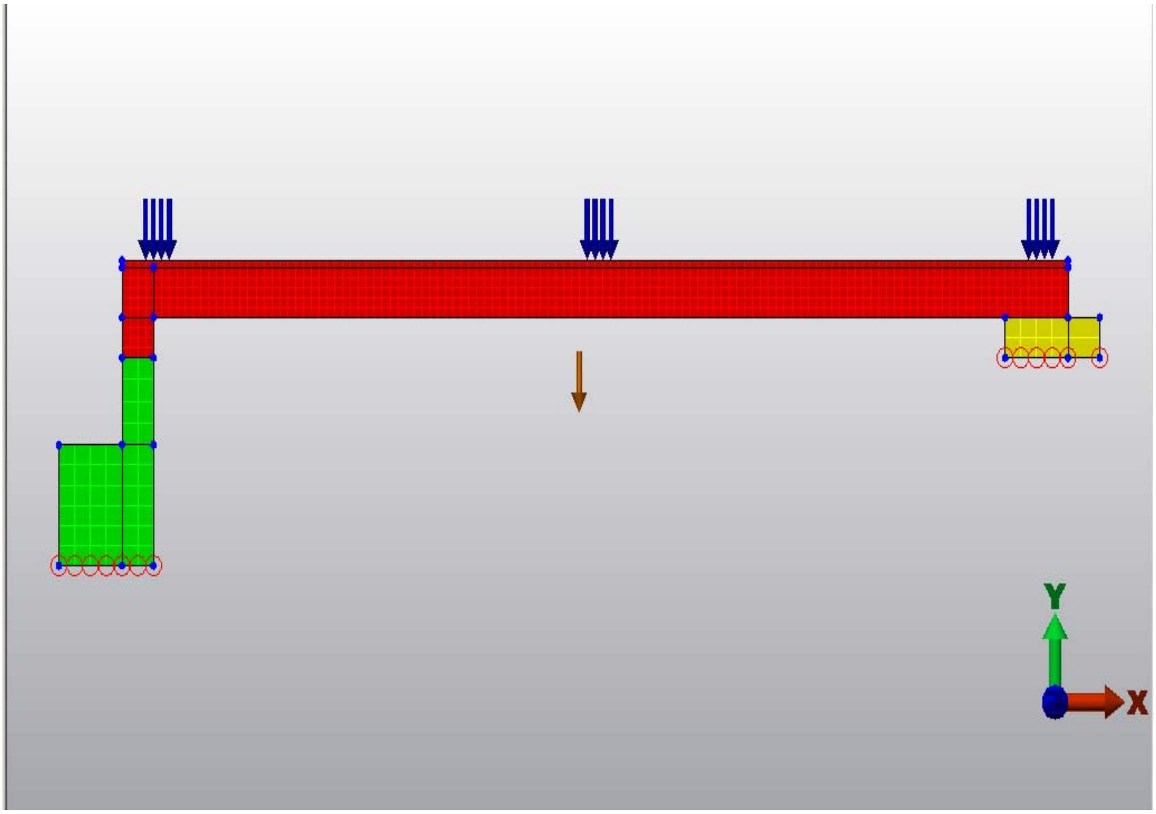

Figure 3.2 – FEM of approach slab without soil.

The contact between the end bent and approach slab was bonded because the top reinforcement runs from approach slab to end bent. Bonded contact is applicable to all element types. The two surfaces will be in perfect contact throughout the analysis when bonded, and the loads are transmitted from one part to the adjacent part. In a stress analysis, when a node on one surface deflects, the node on the adjoining surface will deflect the same amount in the same direction (Autodesk Algor Simulation User's Guide).

Approach slab is placed over the sleeper slab so a surface contact was considered between them with a coefficient of static friction as 0.75. Surface contact is created only if the gap between the parts is zero. If the Surface command is selected, a zero-length contact element is placed between the nodes. The nodes will be free to move away from each other, but the nodes cannot pass through each other when they come into contact. Imagine a very small line created between the nodes on these surfaces. If that line becomes longer during the analysis, it will have no effect on the model. If that line becomes zero length, it will act as a spring with a stiffness value that will resist this motion. Friction can also be added to a contact pair (Autodesk Algor Simulation User's Guide).

The support conditions were fixed for the (Finite Element Model) FEM model with soil. This will be quite close to the real conditions as the soil which is below the end bent would supposedly be immovable. The soil in the upper layers just below the approach slab would move out due to water drainage or high stresses but the soil in lower layers will be in contact with each other and will act as in a fixed condition. The support conditions were assumed to be simply supported slab for the (Finite Element Model) FEM model without soil underneath for this research. The support conditions would affect the results as the moments produced in a simple support are different from a fixed support. This would produce more realistic solution with a higher impact on the slab thus resulting in more improved design of the approach slab.

Linear Stress Analysis and MES Analysis were run in ALGOR and values for deflections and moments were dithered at various parts for the model. The maximum values of deflection for models with soil and without soil were 0.057 in. and 0.179 in., respectively as shown in Fig. 3.3 and Fig. 3.4. Deflections rise approximately three times when the soil moves out which can cause cracks in the structure and would cause a bumpy ride for a driver.

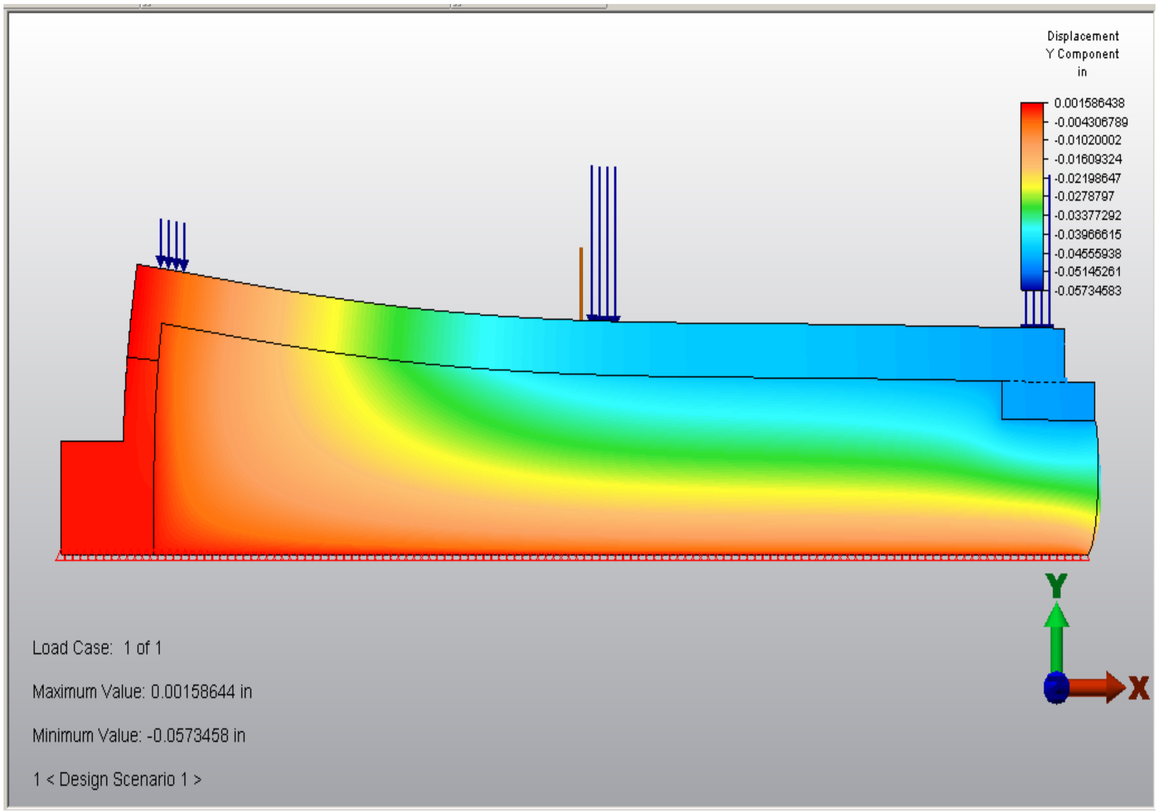

Figure 3.3 – FEM model with soil dithered on displacement.

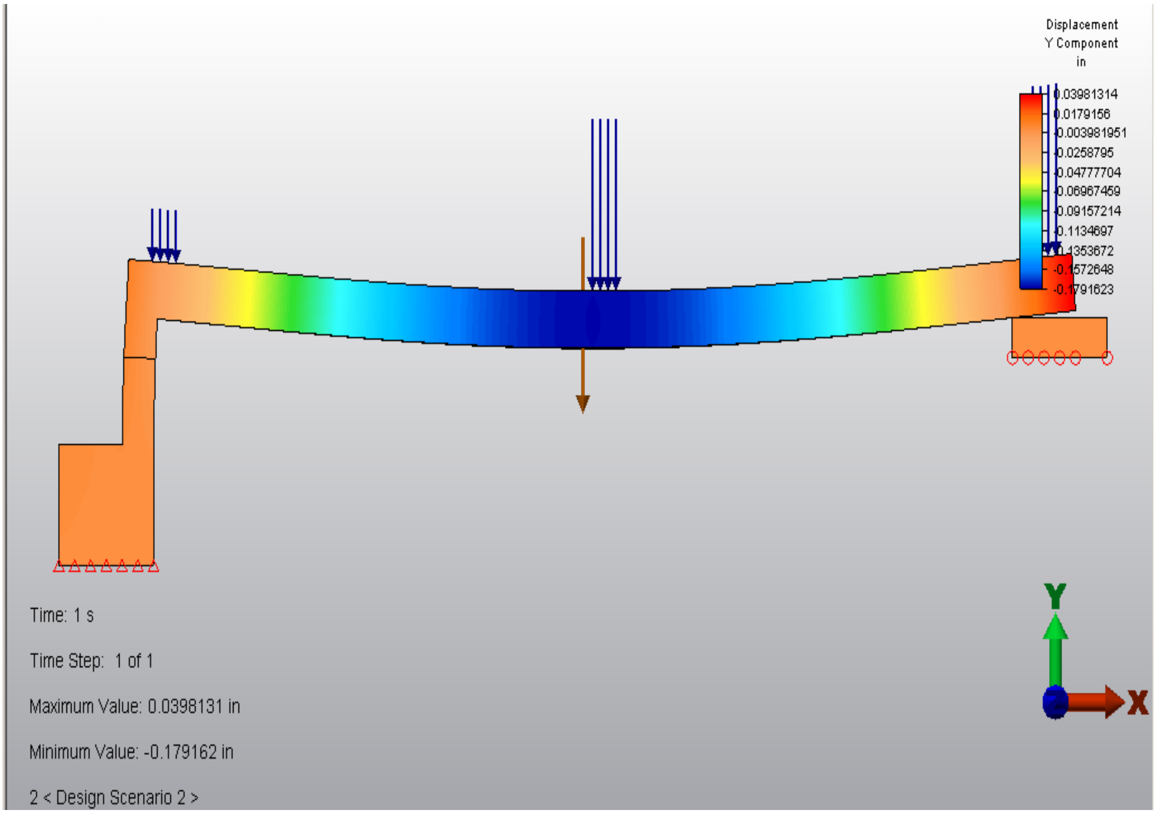

Figure 3.4 – FEM model without soil dithered on displacement.

Bending stress is another important factor which has to be evaluated. The maximum value for bending stress, for the bottom reinforcement, in case of approach slab with soil underneath was 569.245 lbf/in^2 as shown in Fig. 3.5 while that of slab without soil underneath was found to be 3028.978 lbf/in^2 as shown in Fig. 3.6.

Analysis performed using finite element method gives more accurate results and thus it helps in improving the design. The results obtained show that the deflections increase significantly when soil moves out under the approach slab. This will increase the impact of the loads on the slab and thus reducing its stability. Due to increase in moment the slab takes a concave shape which result in the formation of bumps. The situation worsens with time and thus State Departments have to spend a lot of money on its maintenance. This research focuses on developing a design for the slab using the bending stress values obtained on slab without soil underneath it.

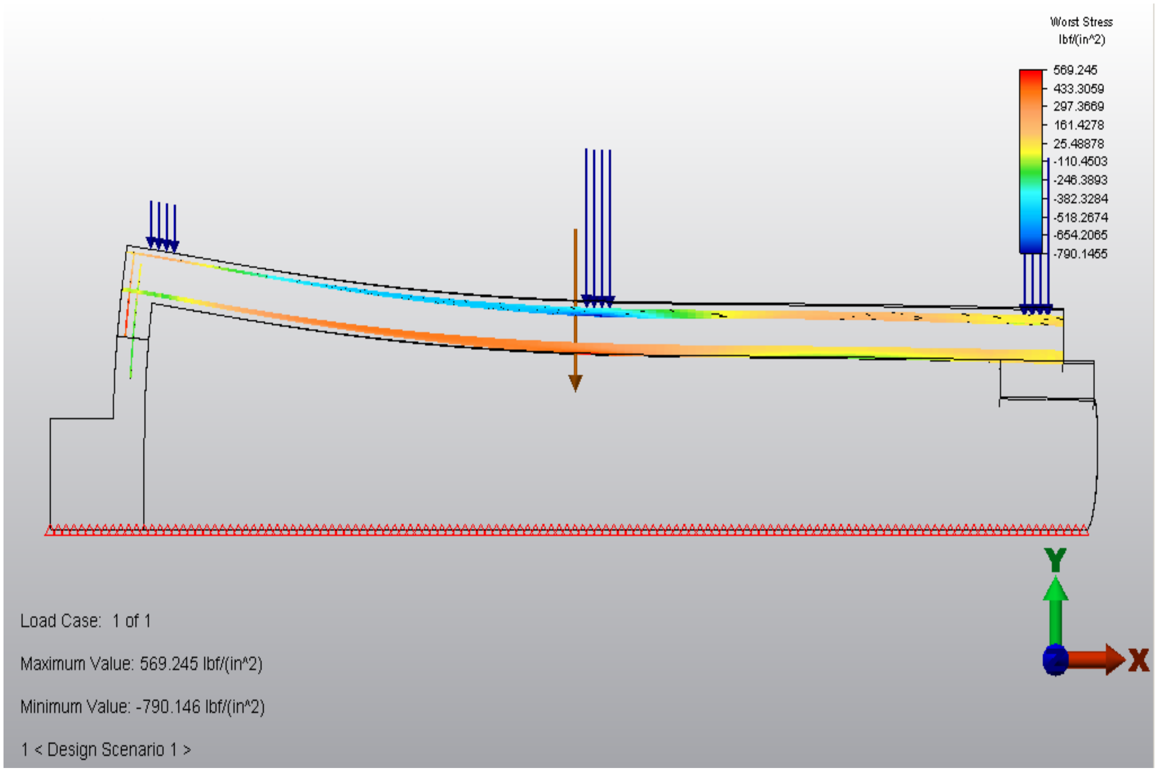

Figure 3.5 – FEM model with soil dithered on beam stress.

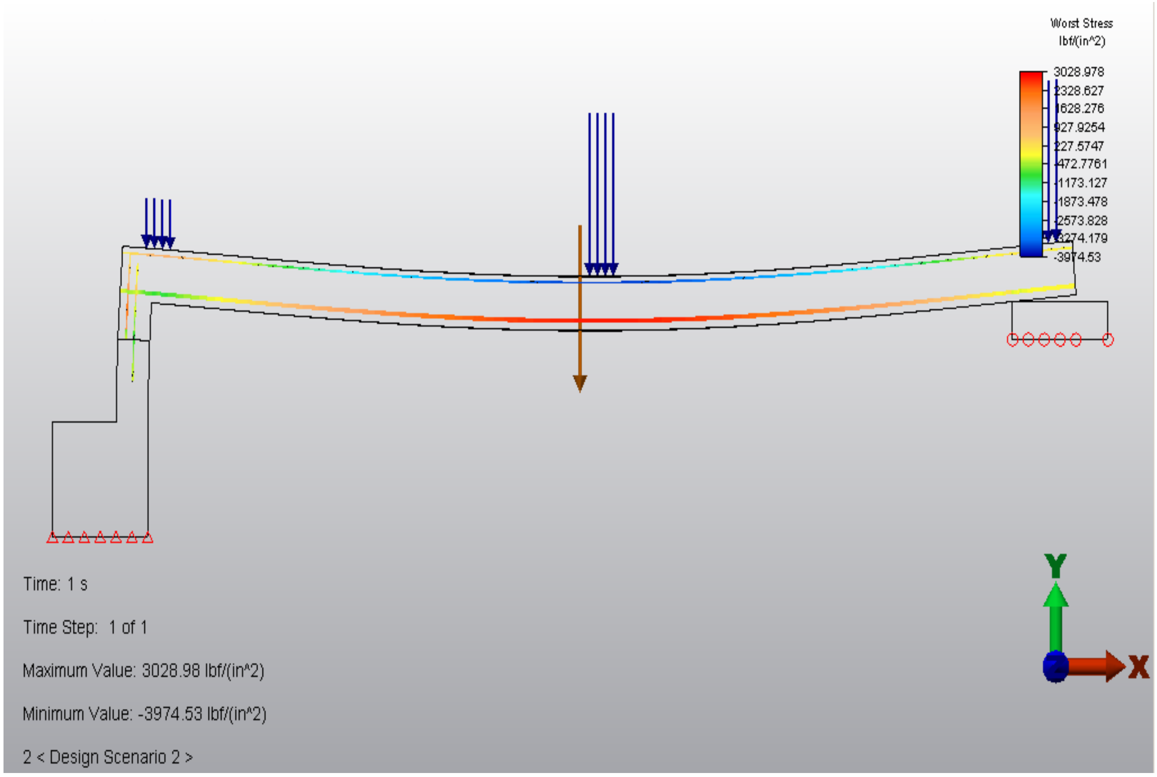

Figure 3.6 – FEM model without soil dithered on beam stress.

An approach slab is designed using the value of bending moment which is obtained by calculating the maximum value of bending stresses. Thus, when the soil moves out beneath the slab, it still has the strength to withstand the loads.

CHAPTER 4

RESULTS AND DISCUSSIONS

Approach slab drawings of different State Departments of Transportation (DOTs) were studied and analyzed. Applied moment and internal moment capacity of the approach slabs for these states were calculated and compared. The calculations were performed considering the approach slab as a simply supported double reinforced beam. One foot width of beam was considered for calculations. The calculations were performed using MathCAD and it was found that the approach slabs for the states of Florida and Michigan were under-designed or strength deficient. Table 4.1 shows the approach slab designs in different State Department of Transportation.

Table 4.1 – Approach Slab Designs in different state DOTs

State	L_{min} (ft.)	h (in.)	$f_{c'}$ (ksi)	A_s (in^2/ft)	$A_{s'}$ (in^2/ft)	d' (in.)	C_c (in.)	ΦM_n (kip*ft/ft)	M_u (kip*ft/ft)
AZ	15	12	3	1.053	0.133	2.5	3	37.57	9.77
FL*	30	12	4.5	1.053	0.310	2.5	4	31.05	80.03
IN	20	10	4	0.630	0.203	2.5	2	19.14	30.16
KY	25	17	3.5	1.580	0	NP	3	90.10	61.72
MI*	20	12	4.5	0.895	0.895	3	3	21.87	31.72
OH	30	17	4.5	2.345	0.207	3	3	129.81	90.40
PA	25	16	3.5	1.693	0.310	2.5	3	85.22	60.50

* denotes that applied moment is more than internal moment capacity; NP means Not Provided

As discussed in previous chapter, finite element analysis of approach slab according to Ohio Department of Transportation specifications was performed and the result for bending stresses was dithered. The bending stress for the bottom reinforcement of approach slab when soil has completely moved out was 3,028.97 lbf/in^2. Maximum applied moment was calculated using the value of bending stress obtained from ALGOR (calculations are shown on the next page) which was found to be 139.90 kip-ft whereas the internal moment capacity of approach slab calculated using ODOT (Ohio Department of Transportation) was 129.81 kip-ft. Therefore, slab is under-designed after all the soil moves out beneath the slab and the slab should be redesigned, calculations for which are done in Appendix A.

CHAPTER 5

RECOMMENDATIONS AND CONCLUSIONS

The two major causes for the formation of bumps are movement/settlement of soil underneath the approach slabs and strength deficient approach slabs. Clayey soil exhibits long-term settlement which is hard to calculate. In the clayey soil regions, the soil should be replaced with granular soil. Proper compaction of the granular should be obtained by following methods given below:

- pneumatic rubber-tired rollers
- vibratory rollers and hand
- handheld rollers

The soil near the end-bent is hard to compact and special considerations should be taking while compacting.

Approach slab was designed with internal moment capacity (ΦM_n) greater than 139.90 kip-ft. Recommendations for approach slabs in Ohio are:

- Length = 30 ft.
- Width = 20 ft.
- Bottom Reinforcement: #10 @ 5.5 in c/c
- Top Reinforcement: #5 @ 18 in c/c

APPENDIX A

Calculations for maximum moment of approach slab from ALGOR

A_s = 38 #10

= 38 * 1.27 = 48.26 in^2

$A_{s'}$ = 12 #5

= 12 * 0.31 = 3.27 in^2

c = 3.81 in (calculated from MathCAD)

B = 20 ft = (20*12) in

h = 17 in

$d^{'}$ = 3 in

d = 14 in

(d – c) = (14 – 3.81) = 10.19 in

(c – $d^{'}$) = (3.81 – 3) = 3 in

Modulus of Elasticity of concrete (E_c) = 33000 * k_1 * $w_c^{1.5}$ * $f_{c'}^{0.5}$

= 33000 * 1 * $0.15^{1.5}$ * $4.5^{0.5}$

= 4066.84 ksi

Modulus of Elasticity of steel (E_s) = 29000 ksi

n = E_s/E_c = 29000/4066.84

n = 7

Moment of inertia (I) = $(B*c^3)/3 + (2n – 1) * A_{s'} * (c – d^{'})^2 + n * A_s * (d – y)^2$

= $(20*12*3.81^3) + (2*7 – 1)*3.72*0.81^2 + 7*48.26*10.19^2$

= 39534.15 in^4

Bending stress from ALGOR (f_s) = 3028.97 lbf/in^2

Maximum moment (M_s) = (f_s * I)/n*(d-c)

= (3028.97 * 39534.15) / (7 * 10.19 * 12000)

= 139.90 kip-ft

Calculations for the design of recommended ODOT Approach Slab

Assume bottom reinforcement of #10 bars @ 5.5 in c/c

Assume top reinforcement of #5 bars @18 in c/c

Beam dimension:

$b := 12 \cdot in$

$h := 17 \cdot in$

$C_c := 3 \cdot in$

$d' := 3 \cdot in$

$d := h - C_c$

$A_{s'} := 0.207 \cdot in^2$

$A_s := 2.77 \cdot in^2$

$W_s := 1.75 \cdot in$

$L := 30 \cdot ft$

Material properties:

$\gamma_c := 0.15 \cdot \frac{kip}{ft^3}$

$f_y := 60 \cdot ksi$

$E := 29000\,ksi$

$f_c := 4.5 \cdot ksi$

$\phi := 0.9$

Assume $c := 1 \cdot in$

Given $A_s \cdot f_y = (0.85 \cdot f_c \cdot 0.85 \cdot c \cdot b) + A_{s'} \cdot \frac{(c - d')}{c} \cdot 0.003\,E$

$c := Find(c)$ $\quad c = 4.13 \cdot in$

$a := 0.85 \cdot c$ $\quad a = 3.51 \cdot in$

$f_{s'} := \frac{|c - d'|}{c} \cdot 0.003\,E$ $\quad f_{s'} = 23.85\,ksi$

CheckCompressionSteel := if($f_{s'} < f_y$, "Compression Steel Not Yielding" , "Compression Steel Yielding")

CheckCompressionSteel = "Compression Steel Not Yielding"

$A_{s2} := \frac{A_{s'} \cdot f_{s'}}{f_y}$ $\quad A_{s2} = 0.08\,in^2$

$A_{s1} := A_s - A_{s2}$ $\quad A_{s1} = 2.69\,in^2$

$\varepsilon_t := \frac{d - c}{c} \cdot 0.003$ $\quad \varepsilon_t = 0.00716$

$M_{LL} := 0.1 \cdot 16.8\,kip \cdot (0.5 \cdot L - 1 \cdot ft) + 0.1 \cdot 24.8\,kip \cdot (0.5 \cdot L - 14 \cdot ft)$

$M_{DL} := 0.125\,b \cdot (h) \cdot \gamma_c \cdot L^2$

$M_{LL} = 26 \cdot kip \cdot ft$

$M_{DL} = 23.91 \cdot kip \cdot ft$

$CheckTensionSteel := if(\varepsilon_t > 0.005, \text{"OK, Tension Steel Yielding Governs Failure"}, \text{"No Good,Revise Section"})$

$CheckTensionSteel = \text{"OK, Tension Steel Yielding Governs Failure"}$

Applied bending moment per meter width of approach slab,

$M_u := (1.25 \cdot M_{DL} + 1.75 \cdot 1.33 \cdot M_{LL})$ $\qquad M_u = 90.4 \cdot kip \cdot ft$

Internal moment capacity per meter width of approach slab,

$\phi M_n := \phi \cdot [A_{s1} \cdot f_y \cdot (d - 0.5 \cdot a) + A_{s2} \cdot f_{s'} \cdot (d - d')]$

$\phi M_n = 149.7 \cdot kip \cdot ft$

Internal moment capacity = 149.7 kip-ft > Applied moment (from ALGOR calculations) = 139.90 kip-ft.

This proves that the design is safe.

USE #10 @ 5.5 in c/c

5 @ 18 in c/c

Moment calculations for AZ DOT drawings:

Beam dimension:

$b := 12 \cdot in$

$h := 12 \cdot in$

$C_c := 3 \cdot in$

$d' := 2.5 \cdot in$

$d := h - C_c$

Material properties:

$A_{s'} := 0.133 \, in^2$

$A_s := 1.053 \, in^2$

$W_s := 1.75 \, in$

$L := 15 \cdot ft$

$\gamma_c := 0.15 \cdot \frac{kip}{ft^3}$

$E := 29000 \, ksi$

$\phi := 0.9$

$f_y := 60 \cdot ksi$

$f_c := 3 \cdot ksi$

Assume $c := 1 \cdot mm$

Given $A_s \cdot f_y = \left(0.85 f_c \cdot 0.85 c \cdot b\right) + A_{s'} \cdot \frac{(c - d')}{c} \cdot 0.003 E$

$c := \text{Find}(c)$ $\quad c = 2.44 \, in$

$a := 0.85 c$ $\quad a = 2.07 \cdot in$

$f_{s'} := \frac{|c - d'|}{c} \cdot 0.003 E$ $\quad f_{s'} = 2.14 \, ksi$

CheckCompressionSteel := if($f_{s'} < f_y$, "Compression Steel Not Yielding" , "Compression Steel Yielding")

CheckCompressionSteel = "Compression Steel Not Yielding"

$A_{s2} := \frac{A_{s'} \cdot f_{s'}}{f_y}$ $\quad A_{s2} = 4.74 \times 10^{-3} \cdot in^2$

$M_{LL} := 0.1 \cdot (32 \cdot kip) \cdot (0.5 \cdot L - 7 \cdot ft)$ $\quad M_{LL} = 1.6 \, kip \cdot ft$

$A_{s1} := A_s - A_{s2}$ $\quad A_{s1} = 1.05 \, in^2$

$M_{DL} := 0.125 \, b \cdot \left(h + W_s\right) \cdot \gamma_c \cdot L^2$ $\quad M_{DL} = 4.83 \, kip \cdot ft$

$\varepsilon_t := \frac{d - c}{c} \cdot 0.003$ $\quad \varepsilon_t = 0.00807$

CheckTensionSteel := if($\varepsilon_t > 0.005$, "OK, Tension Steel Yielding Governs Failure" , "No Good,Revise Section")

CheckTensionSteel = "OK, Tension Steel Yielding Governs Failure"

Applied bending moment per meter width of approach slab,

$M_u := (1.25 \cdot M_{DL} + 1.75 \cdot 1.33 \cdot M_{LL})$ $\qquad M_u = 9.77 \cdot kip \cdot ft$

Internal moment capacity per meter width of approach slab,

$\phi M_n := \phi \cdot \left[A_{s1} \cdot f_y \cdot (d - 0.5 \cdot a) + A_{s2} \cdot f_{s'} \cdot (d - d')\right]$

$\phi M_n = 37.57 \cdot kip \cdot ft$

Moment calculations for FL DOT drawings:

Beam dimension:

$b := 12 \cdot in$

$h := 12 \cdot in$

$C_c := 4 \cdot in$

$d' := 2.5 \cdot in$

$d := h - C_c$

Material properties:

$A_{s'} := 0.31 \cdot in^2$

$A_s := 1.053\, in^2$

$W_s := 1.75 \cdot in$

$L := 30 \cdot ft$

$\gamma_c := 0.15 \cdot \frac{kip}{ft^3}$

$E := 29000\, ksi$

$\phi := 0.9$

$f_y := 60 \cdot ksi$

$f_c := 4.5 \cdot ksi$

Assume $c := 1 \cdot in$

Given $A_s \cdot f_y = \left(0.85 \cdot f_c \cdot 0.85 \cdot c \cdot b\right) + A_{s'} \cdot \frac{(c - d')}{c} \cdot 0.003\, E$

$c := \text{Find}(c)$ $\qquad c = 1.86 \cdot in$

$a := 0.85 \cdot c$ $\qquad a = 1.58 \cdot in$

$f_{s'} := \frac{|c - d'|}{c} \cdot 0.003\, E$ $\qquad f_{s'} = 30.05\, ksi$

CheckCompressionSteel := if($f_{s'} < f_y$, "Compression Steel Not Yielding" , "Compression Steel Yielding")

CheckCompressionSteel = "Compression Steel Not Yielding"

$A_{s2} := \frac{A_{s'} \cdot f_{s'}}{f_y}$ $\qquad A_{s2} = 100.17\, mm^2$ $\qquad M_{LL} := 0.1 \cdot (71171.5\, N) \cdot (0.5 \cdot L)$ $\qquad M_{LL} = 24 \cdot kip \cdot ft$

$A_{s1} := A_s - A_{s2}$ $\qquad A_{s1} = 579.18\, mm^2$ $\qquad M_{DL} := 0.125\, b \cdot \left(h + W_s\right) \cdot \gamma_c \cdot L^2$ $\qquad M_{DL} = 19.34\, kip \cdot ft$

$\varepsilon_t := \frac{d - c}{c} \cdot 0.003$ $\qquad \varepsilon_t = 0.00992$

CheckTensionSteel := if($\varepsilon_t > 0.005$, "OK, Tension Steel Yielding Governs Failure" , "No Good,Revise Section")

CheckTensionSteel = "OK, Tension Steel Yielding Governs Failure"

Applied bending moment per meter width of approach slab,

$M_u := \left(1.25 \cdot M_{DL} + 1.75 \cdot 1.33 \cdot M_{LL}\right)$ $\qquad M_u = 80.03 \, kip \cdot ft$

Internal moment capacity per meter width of approach slab,

$\phi M_n := \phi \cdot \left[A_{s1} \cdot f_y \cdot (d - 0.5 \cdot a) + A_{s2} \cdot f_{s'} \cdot (d - d')\right]$

$\phi M_n = 31.05 \, kip \cdot ft$

Moment calculations for IN DOT drawings:

Beam dimension:

$b := 12 \cdot in$

$h := 10 \cdot in$

$C_c := 2 \cdot in$

$d' := 2.5 \cdot in$

$d := h - C_c$

$A_{s'} := 0.203 \, in^2$

$A_s := 0.630 \, in^2$

$W_s := 1.75 \, in$

$L := 20 \cdot ft$

Material properties:

$\gamma_c := 0.15 \cdot \frac{kip}{ft^3}$

$f_y := 60 \cdot ksi$

$E := 29000 \, ksi$

$f_c := 4 \cdot ksi$

$\phi := 0.9$

Assume $\quad c := 1 \cdot in$

Given $\quad A_s \cdot f_y = (0.85 \, f_c \cdot 0.85 \, c \cdot b) + A_{s'} \cdot \frac{(c - d')}{c} \cdot 0.003 \, E$

$c := \text{Find}(c) \qquad c = 1.46 \, in$

$a := 0.85 \, c \qquad a = 1.24 \, in$

$f_{s'} := \frac{|c - d'|}{c} \cdot 0.003 \, E \qquad f_{s'} = 62.44 \, ksi$

CheckCompressionSteel := if$\left(f_{s'} < f_y, \text{"Compression Steel Not Yielding"}, \text{"Compression Steel Yielding"}\right)$

CheckCompressionSteel = "Compression Steel Yielding"

$A_{s2} := \frac{A_{s'} \cdot f_{s'}}{f_y} \qquad A_{s2} = 0.21 \cdot in^2 \qquad M_{LL} := 0.1 \cdot (32 \cdot kip) \cdot (0.5 \, L - 7 \cdot ft)$

$A_{s1} := A_s - A_{s2} \qquad A_{s1} = 0.42 \, in^2 \qquad M_{DL} := 0.125 \, b \cdot (h) \cdot \gamma_c \cdot L^2 \qquad M_{LL} = 9.6 \, kip \cdot ft$

$\varepsilon_t := \frac{d - c}{c} \cdot 0.003 \qquad \varepsilon_t = 0.01349 \qquad M_{DL} = 6.25 \, kip \cdot ft$

CheckTensionSteel := if$\left(\varepsilon_t > 0.005, \text{"OK, Tension Steel Yielding Governs Failure"}, \text{"No Good,Revise Section"}\right)$

CheckTensionSteel = "OK, Tension Steel Yielding Governs Failure"

Applied bending moment per meter width of approach slab,

$M_u := (1.25 \cdot M_{DL} + 1.75 \cdot 1.33 \cdot M_{LL})$ $\qquad M_u = 30.16\,kip \cdot ft$

Internal moment capacity per meter width of approach slab,

$\phi M_n := \phi \cdot [A_{s1} \cdot f_y \cdot (d - 0.5 \cdot a) + A_{s2} \cdot f_y \cdot (d - d')]$

$\phi M_n = 19.14\,kip \cdot ft$

Moment calculations for KY DOT drawings:

Beam dimension:

$b := 12 \cdot in$

$h := 17 \cdot in$

$C_c := 3 \cdot in$

$d' := 0 \cdot in$

$d := h - C_c$

$A_{s'} := 0 \cdot in^2$

$A_s := 1.58 \cdot in^2$

$W_s := 1.75 \cdot in$

$L := 25 \cdot ft$

Material properties:

$\gamma_c := 0.15 \cdot \frac{kip}{ft^3}$

$f_y := 60 \cdot ksi$

$E := 29000 \, ksi$

$f_c := 3.5 \cdot ksi$

$\phi := 0.9$

Assume $c := 1 \cdot in$

Given $A_s \cdot f_y = (0.85 \cdot f_c \cdot 0.85 \cdot c \cdot b) + A_{s'} \cdot \frac{(c - d')}{c} \cdot 0.003 \cdot E$

$c := \text{Find}(c)$ $\quad c = 3.12 \cdot in$

$a := 0.85 \cdot c$ $\quad a = 2.66 \cdot in$

$f_{s'} := \frac{|c - d'|}{c} \cdot 0.003 \cdot E$ $\quad f_{s'} = 87 \cdot ksi$

CheckCompressionSteel := if($f_{s'} < f_y$, "Compression Steel Not Yielding" , "Compression Steel Yielding")

CheckCompressionSteel = "Compression Steel Yielding"

$A_{s2} := \frac{A_{s'} \cdot f_{s'}}{f_y}$ $\quad A_{s2} = 0 \cdot in^2$ $\quad M_{LL} := 0.1 \cdot (32 \cdot kip) \cdot (0.5 \cdot L - 7 \cdot ft)$

$A_{s1} := A_s - A_{s2}$ $\quad A_{s1} = 1.58 \cdot in^2$ $\quad M_{DL} := 0.125 \cdot b \cdot (h) \cdot \gamma_c \cdot L^2$ $\quad M_{LL} = 17.6 \cdot kip \cdot ft$

$\varepsilon_t := \frac{d - c}{c} \cdot 0.003$ $\quad \varepsilon_t = 0.01044$ $\quad M_{DL} = 16.6 \cdot kip \cdot ft$

CheckTensionSteel := if($\varepsilon_t > 0.005$, "OK, Tension Steel Yielding Governs Failure" , "No Good,Revise Section")

CheckTensionSteel = "OK, Tension Steel Yielding Governs Failure"

Applied bending moment per meter width of approach slab,

$$M_u := \left(1.25 \cdot M_{DL} + 1.75 \cdot 1.33 \cdot M_{LL}\right) \qquad M_u = 61.72 \cdot kip \cdot ft$$

Internal moment capacity per meter width of approach slab,

$$\phi M_n := \phi \cdot \left[A_{s1} \cdot f_y \cdot (d - 0.5 \cdot a) + A_{s2} \cdot f_y \cdot (d - d')\right]$$

$$\phi M_n = 90.1 \cdot kip \cdot ft$$

Moment calculations for MI DOT drawings:

Beam dimension:

$b := 12\cdot in$

$h := 12\cdot in$

$C_c := 3\cdot in$

$d' := 3\cdot in$

$d := h - C_c$

$A_{s'} := 0.895\,in^2$

$A_s := 0.895\,in^2$

$W_s := 1.75\cdot in$

$L := 20\cdot ft$

Material properties:

$\gamma_c := 0.15\cdot\frac{kip}{ft^3}$

$E := 29000\,ksi$

$\phi := 0.9$

$f_y := 60\cdot ksi$

$f_c := 4.5\cdot ksi$

Assume $c := 1\cdot in$

Given $A_s\cdot f_y = \left(0.85\cdot f_c\cdot 0.85\cdot c\cdot b\right) + A_{s'}\cdot\frac{(c-d')}{c}\cdot 0.003\cdot E$

$c := \text{Find}(c)$ $\quad c = 2.16\cdot in$

$a := 0.85\cdot c$ $\quad a = 1.83\cdot in$

$f_{s'} := \frac{|c-d'|}{c}\cdot 0.003\cdot E$ $\quad f_{s'} = 34.02\cdot ksi$

CheckCompressionSteel := if($f_{s'} < f_y$, "Compression Steel Not Yielding" , "Compression Steel Yielding")

CheckCompressionSteel = "Compression Steel Not Yielding"

$A_{s2} := \frac{A_{s'}\cdot f_{s'}}{f_y}$ $\quad A_{s2} = 0.51\cdot in^2$ $\quad M_{LL} := 0.1\cdot(32\cdot kip)\cdot(0.5\cdot L - 7\cdot ft)$

$A_{s1} := A_s - A_{s2}$ $\quad A_{s1} = 0.39\cdot in^2$ $\quad M_{DL} := 0.125\cdot b\cdot(h)\cdot\gamma_c\cdot L^2$ $\quad M_{LL} = 9.6\cdot kip\cdot ft$

$\varepsilon_t := \frac{d-c}{c}\cdot 0.003$ $\quad \varepsilon_t = 0.00952$ $\quad M_{DL} = 7.5\cdot kip\cdot ft$

CheckTensionSteel := if($\varepsilon_t > 0.005$, "OK, Tension Steel Yielding Governs Failure" , "No Good,Revise Section")

CheckTensionSteel = "OK, Tension Steel Yielding Governs Failure"

Applied bending moment per meter width of approach slab,

$$M_u := \left(1.25 \cdot M_{DL} + 1.75 \cdot 1.33 \cdot M_{LL}\right) \qquad M_u = 31.72 \cdot kip \cdot ft$$

Internal moment capacity per meter width of approach slab,

$$\phi M_n := \phi \cdot \left[A_{s1} \cdot f_y \cdot (d - 0.5 \cdot a) + A_{s2} \cdot f_{s'} \cdot (d - d')\right]$$

$$\phi M_n = 21.87 \cdot kip \cdot ft$$

Moment calculations for OH DOT drawings:

Beam dimension:

$b := 12 \cdot in$

$h := 17 \cdot in$

$C_c := 3 \cdot in$

$d' := 3 \cdot in$

$d := h - C_c$

$A_{s'} := 0.207 \cdot in^2$

$A_s := 2.345 \, in^2$

$W_s := 1.75 \, in$

$L := 30 \cdot ft$

Material properties:

$\gamma_c := 0.15 \cdot \frac{kip}{ft^3}$

$E := 29000 \, ksi$

$\phi := 0.9$

$f_y := 60 \cdot ksi$

$f_c := 4.5 \cdot ksi$

Assume $c := 1 \cdot in$

Given $A_s \cdot f_y = (0.85 f_c \cdot 0.85 c \cdot b) + A_{s'} \cdot \frac{(c - d')}{c} \cdot 0.003 E$

$c := \text{Find}(c)$ $\quad c = 3.54 \, in$

$a := 0.85 c$ $\quad a = 3.01 \cdot in$

$f_{s'} := \frac{|c - d'|}{c} \cdot 0.003 E$ $\quad f_{s'} = 13.19 \, ksi$

CheckCompressionSteel := if($f_{s'} < f_y$, "Compression Steel Not Yielding" , "Compression Steel Yielding")

CheckCompressionSteel = "Compression Steel Not Yielding"

$A_{s2} := \frac{A_{s'} \cdot f_{s'}}{f_y}$ $\quad A_{s2} = 0.05 \, in^2$

$M_{LL} := 0.1 \cdot 16.8 \, kip \cdot (0.5 \cdot L - 1 \cdot ft) + 0.1 \cdot 24.8 \, kip \cdot (0.5 \cdot L - 14 \cdot ft)$

$A_{s1} := A_s - A_{s2}$ $\quad A_{s1} = 2.3 \cdot in^2$

$M_{DL} := 0.125 b \cdot (h) \cdot \gamma_c \cdot L^2$

$M_{LL} = 26 \cdot kip \cdot ft$

$\varepsilon_t := \frac{d - c}{c} \cdot 0.003$ $\quad \varepsilon_t = 0.00888$

$M_{DL} = 23.91 \cdot kip \cdot ft$

CheckTensionSteel := if($\varepsilon_t > 0.005$, "OK, Tension Steel Yielding Governs Failure" , "No Good,Revise Section")

CheckTensionSteel = "OK, Tension Steel Yielding Governs Failure"

Applied bending moment per meter width of approach slab,

$$M_u := \left(1.25 \cdot M_{DL} + 1.75 \cdot 1.33 \cdot M_{LL}\right) \qquad M_u = 90.4 \cdot kip \cdot ft$$

Internal moment capacity per meter width of approach slab,

$$\phi M_n := \phi \cdot \left[A_{s1} \cdot f_y \cdot (d - 0.5 \cdot a) + A_{s2} \cdot f_{s'} \cdot (d - d')\right]$$

$$\phi M_n = 129.81 \cdot kip \cdot ft$$

Moment calculations for PA DOT drawings:

Beam dimension:

$b := 12 \cdot in$

$h := 16 \cdot in$

$C_c := 3 \cdot in$

$d' := 2.5 \cdot in$

$d := h - C_c$

$A_{s'} := 0.31 \cdot in^2$

$A_s := 1.693 \, in^2$

$W_s := 1.75 \cdot in$

$L := 25 \cdot ft$

Material properties:

$\gamma_c := 0.15 \cdot \frac{kip}{ft^3}$

$f_y := 60 \cdot ksi$

$E := 29000 \, ksi$

$f_c := 3.5 \cdot ksi$

$\phi := 0.9$

Assume $c := 1 \cdot in$

Given $A_s \cdot f_y = \left(0.85 \cdot f_c \cdot 0.85 \cdot c \cdot b\right) + A_{s'} \cdot \frac{(c - d')}{c} \cdot 0.003 \, E$

$c := \mathrm{Find}(c)$ $c = 3.16 \cdot in$

$a := 0.85 \cdot c$ $a = 2.69 \cdot in$

$f_{s'} := \frac{|c - d'|}{c} \cdot 0.003 \, E$ $f_{s'} = 18.2 \cdot ksi$

CheckCompressionSteel := if($f_{s'} < f_y$, "Compression Steel Not Yielding" , "Compression Steel Yielding")

CheckCompressionSteel = "Compression Steel Not Yielding"

$A_{s2} := \frac{A_{s'} \cdot f_{s'}}{f_y}$ $A_{s2} = 0.09 \cdot in^2$ $M_{LL} := 0.1 \cdot (32 \cdot kip) \cdot (0.5 \cdot L - 7 \cdot ft)$

$A_{s1} := A_s - A_{s2}$ $A_{s1} = 1.6 \cdot in^2$ $M_{DL} := 0.125 \, b \cdot (h) \cdot \gamma_c \cdot L^2$ $M_{LL} = 17.6 \, kip \cdot ft$

$\varepsilon_t := \frac{d - c}{c} \cdot 0.003$ $\varepsilon_t = 0.00934$ $M_{DL} = 15.62 \cdot kip \cdot ft$

CheckTensionSteel := if($\varepsilon_t > 0.005$, "OK, Tension Steel Yielding Governs Failure" , "No Good,Revise Section")

CheckTensionSteel = "OK, Tension Steel Yielding Governs Failure"

Applied bending moment per meter width of approach slab,

$M_u := (1.25 \cdot M_{DL} + 1.75 \cdot 1.33 \cdot M_{LL})$ $\qquad$ $M_u = 60.5 \cdot kip \cdot ft$

Internal moment capacity per meter width of approach slab,

$\phi M_n := \phi \cdot \left[A_{s1} \cdot f_y \cdot (d - 0.5 \cdot a) + A_{s2} \cdot f_{s'} \cdot (d - d')\right]$

$\phi M_n = 85.22 \cdot kip \cdot ft$

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