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16. Abstract
Researchers propose a new approach slab, which has a one-span slab after reviewing the existing knowledge and the existing practice, performing numerical analyses, and conducting model scale simulations. Some of the most important conclusions from the numerical analyses are listed below:

1. The presence of the abutment wall founded on piles creates a difference in settlement between the abutment wall and the embankment.
2. This differential settlement is drastically reduced in the absence of the wall.
3. The transition zone is about 40 ft with 80 percent of the maximum settlement occurring in the first 20 ft for a uniform load case.
4. The size of the sleeper slab and support slab influences the settlement of the slab when load is applied to the slab. The optimum width of both slabs is about 5 ft. The height of the embankment is influencing the settlement of the embankment.

Based on the work done in this research project, the new recommended approach slab is at least 20 ft long and has one span from the abutment to the sleeper slab. It should be designed to carry the full traffic load without support on the soil except at both ends; the support slab used in the current solution is removed and the wide flange is kept on the embankment side as a temperature elongation joint. This new approach slab will simplify construction, be less expensive, and place less emphasis on the need for very good compaction close to the abutment wall. The BEST (Bridge to Embankment Simulator of Transition) device, which is a 1/20th scale model of the typical transition, was used to simulate the existing approach slab and the new proposed approach slabs. The results indicate the following:

1. The proposed new approach slab (one-span) with a 20 ft simulated span length gave a smaller bump than the current two-slab approach slab.
2. The soil with the higher compaction level developed less bump at the sleeper slab than the lower compaction soil.
3. The bump size increased with the number of cycles in a straight line on a log-log plot.

The maximum vertical acceleration of the simulated car was 32 m/sec^2 at a velocity of 13.78 km/hr.

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INVESTIGATION OF SETTLEMENT AT BRIDGE APPROACH SLAB EXPANSION JOINT: NUMERICAL SIMULATIONS AND MODEL TESTS

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ACKNOWLEDGMENTS

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CHAPTER 1:
INTRODUCTION

A settlement at the bridge approach slab expansion joint often develops at the end of a bridge near the interface between the abutment and the embankment. The approach slabs are reinforced concrete slabs that are used to span problematic areas between the pavement and the bridge abutment. The problem of the bump at the end of the bridge exists at 25 percent of all bridges in Texas. It is estimated that TxDOT spends 7.0 million dollars each year for the maintenance associated with the problem of the bump at the end of the bridge. This number is based on the results of the survey completed during the first year of this project and uses 2001 dollars. This number rises to $100 million for the USA. Also in the United States, 35 percent of bridges are deficient and the cost of repair is estimated at $78 billion (Transportation Builder, 1995).

The public feels those bumps regularly. Reduction in steering response, distraction to the driver, added risk and expense to maintenance operations, and reduction in a transportation agency's public image are all undesirable effects of these uneven and irregular transitions. In spite of all these problems, many state departments of transportation regard the settlement of bridge approach slabs as a substantial maintenance problem, and guidelines affecting the use, design methodology, material specifications, and construction techniques vary greatly from state to state (Hoppe, 1999). The intended function of an approach slab is to (Briaud et al., 1997):

1. span the void that may develop below the slab;
2. prevent slab deflection, which could result in settlement near the abutment;
3. provide a ramp for the differential settlement between the embankment and the abutment (This function is affected by the length of the approach slab and the magnitude of the differential settlement.); and
4. provide a better seal against water percolation and erosion of the embankment.

The bump at the end of the bridge may look like a simple problem at first glance: the embankment settles more than the bridge because embankments on soil compress more than abutment on deep foundations. In fact, the bump at the end of the bridge is a very complex problem including factors such as compaction, drainage, embankment height, traffic level, temperature cycles, and downdrag on the abutment. The first year work is briefly described in Chapter 2. Current construction practices are shown in Chapter 3. ABAQUS was used to simulate the behavior of the transition zone, and the results of this simulation are summarized in Chapter 4. The new approach slab is proposed in Chapter 5. Ten model scale simulations have been carefully done by using the Bridge Embankment Simulator of Transition (BEST) device, which is a 1/20 model scale of the real transition. The test and results are discussed in Chapter 6. Conclusions and possible recommendations for future work are found in Chapter 7.

The primary goals of this project are to investigate the settlement at the bridge approach slab expansion joint, identify the reasons for the differential settlement, define current design and construction problems, and find a way to minimize the bump at the end of the bridge. The second year work was mainly focused on finding a way to minimize the bump at the end of the bridge. The type of approach slab studied was the articulated wide flange approach slab used in the Houston area, many of which have developed dips at the articulation joint with unacceptable ride comfort indices.

CHAPTER 2:
REVIEW OF THE FIRST YEAR WORK

Review of Previous Work

Many reports related to the differential settlement along bridge approach slabs have been published by several departments of transportation and researchers. These reports were studied and summarized in the first year report. While many causes have been identified, the interaction between the cause and the effect remains complex.

Kramer and Sajer (1991) studied the contributing causes of the bump formation. Table 2.1 shows a summary of their findings.

Table 2.1. Causes of Bridge Approach Problems Categorized

(after Kramer and Sajer, 1991).

1. **Differential Settlement**	
Compression of natural soils	Primary consolidation, secondary compression, and creep
Compression of embankment soils	Volume changes and distortional movements/creep of embankment soils
Local compression at bridge/pavement interface	Inadequate compaction at bridge/pavement interface, drainage and erosion problems, rutting/distortion of pavement section, traffic loading, and thermal bridge movements
2. **Movement of Abutments**	
Vertical movement	Settlement of soil beneath, downdrag, erosion of soil beneath and around abutment
Horizontal movement	Excessive lateral pressures, thermal movements, swelling pressures from expansive soils, and lateral deformation of embankment and natural soils
3. **Design/Construction Problems**	
Engineer-related	Improper materials, lift thickness, and compaction requirements
Contractor-related	Improper equipment, overexcavation for abutment construction, and survey/grade errors
Inspector-related/ Poor quality control	Lack of inspection personnel and improper inspection personnel training
Design-related	No provision for bridge expansion/contraction spill-through design resulting in the migration of fill material from behind the abutment

Questionnaire

This task consisted in sending an email and hard copy survey to all the TxDOT districts, collecting the answers, analyzing them, and summarizing them. Some results are shown below. This survey indicated that the problem is widespread (about 25 percent of 21,291 bridges surveyed have a bump problem) and that it is costly (about 7.0 million dollars per year in maintenance and repair for 49,100 bridges in Texas in 2001). Details and a summary of the survey are in the first year report.

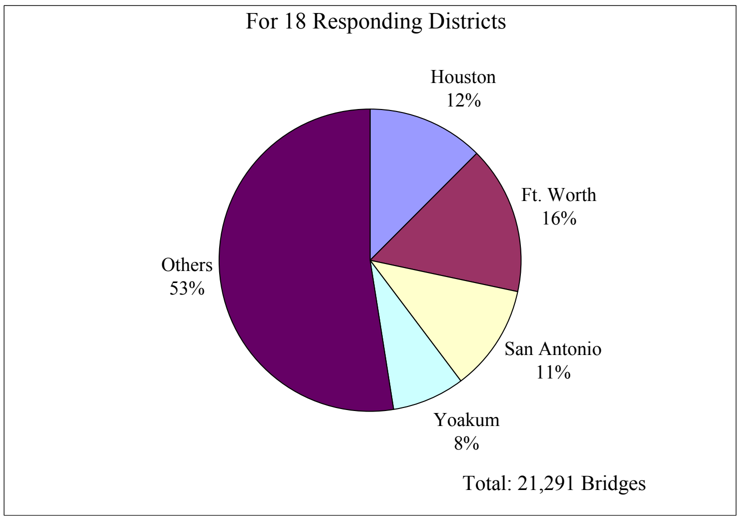


Figure 2.1. Number of Bridges with Bump Problem.

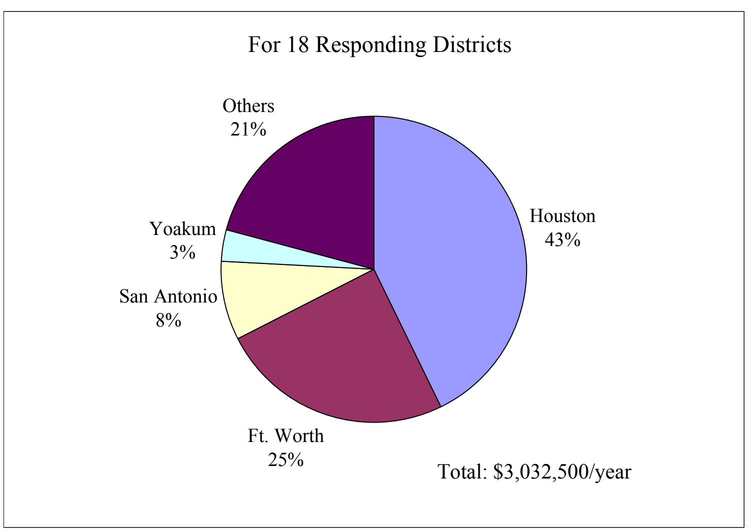


Figure 2.2. Maintenance Cost.

Site Survey

As part of this research work, a total of 18 sites in the TxDOT Houston District were surveyed (Tables 2.2 and 2.3). The methodology for this field survey was simple visual inspection. Among the 18 sites that they visually investigated, researchers classified 10 sites as poor performance locations. The primary factor to classify a test site was the 'bump rating' that was obtained by visual inspection and drive-by survey. Based on this 'bump rating' and other site factors, the researchers selected US290 over FM362 and SH249 at Grant Rd. for the advanced study.

Table 2.2. Bad Performing Locations.

Highway	Highway Intersection	County	Comment
IH45	Almeda Genoa	Harris	Both directions treated with Uretech 3 years ago Approach Embankment: 16'- 17' Bump Scale[1]: 1
BW8	At SH3	Harris	Eastbound treated with Uretech 3 years ago Approach Embankment: 16' - 17' Bump Scale[1]: 1
SH99	At Owens Rd.	Ft. Bend	Approach Embankment: 16' Bump Scale[1]: 0 - 1
SH99	At Oyster Ck.	Ft. Bend	Approach Slab: PCC & 40' Approach Embankment: 10' Bump Scale[1]: 0 - 1
US59	Before Hillcroft exit ramp	Harris	Approach Embankment: 16' - 17' Bump Scale[1]: 1
SH225	Center St. and Rohm-Hass	Harris	Repairs are planned Approach Embankment: 16' - 17' Bump Scale[1]: 1
IH45	At Parker Rd.	Harris	Approach Embankment: 16' - 17' Bump Scale[1]: 1
US59	Saunders/Parker Rd.	Harris	Repaired but still rough Approach Embankment: 16' - 17' Bump Scale[1]: 1
SH249	At Grant Rd.	Harris	Approach Embankment: 16' - 17' Bump Scale[1]: 2
US290	Over FM362	Waller	Repaired but still rough Approach Embankment: 16' - 17' Bump Scale[1]: 1 - 2

[1] Bump Scale is explained in Chapter 5 of the first year report.

Table 2.3. Good Performing Locations.

Highway	Highway Intersection	County	Comment
SH6	At Flat Bank Ck.	Ft. Bend	Approach Slab: PCC & 40' Approach Embankment: 10' Bump Scale[1]: 0
FM1876	At A22 Ditch	Ft. Bend	Approach Slab: PCC & 16' Approach Embankment: 10' Bump Scale[1]: 0
FM1876	At Keegans Bayou	Ft. Bend	Approach Slab: PCC & 16' Approach Embankment: 10' Bump Scale[1]: 0
SH99	At Bullhead Slough	Ft. Bend	Approach Slab: PCC & 16' Approach Embankment: 10' Bump Scale[1]: 0
SH99	At Brazos River	Ft. Bend	Approach Slab: PCC & 17' Approach Embankment: 0' Bump Scale[1]: 0
FM3345	East of FM1092	Ft. Bend	Roadway End of CRCP (not a bridge)
FM3345	West of FM2234	Ft. Bend	Roadway End of CRCP (not a bridge)
FM3345	At Stafford Run	Ft. Bend	Approach Slab: PCC & 16' Approach Embankment: 10' Bump Scale[1]: 0

Site Description

The approach slabs at both sites are two-span approach slab (2SAS) with a wide flange beam (Figures 2.3 and 2.4). Figure 2.5 shows the cross sections of US290 over FM362 and SH249 at Grant Rd. Circled numbers on Figures 2.3 and 2.4 indicate the bump scale explained in Chapter 5 of the first year report.

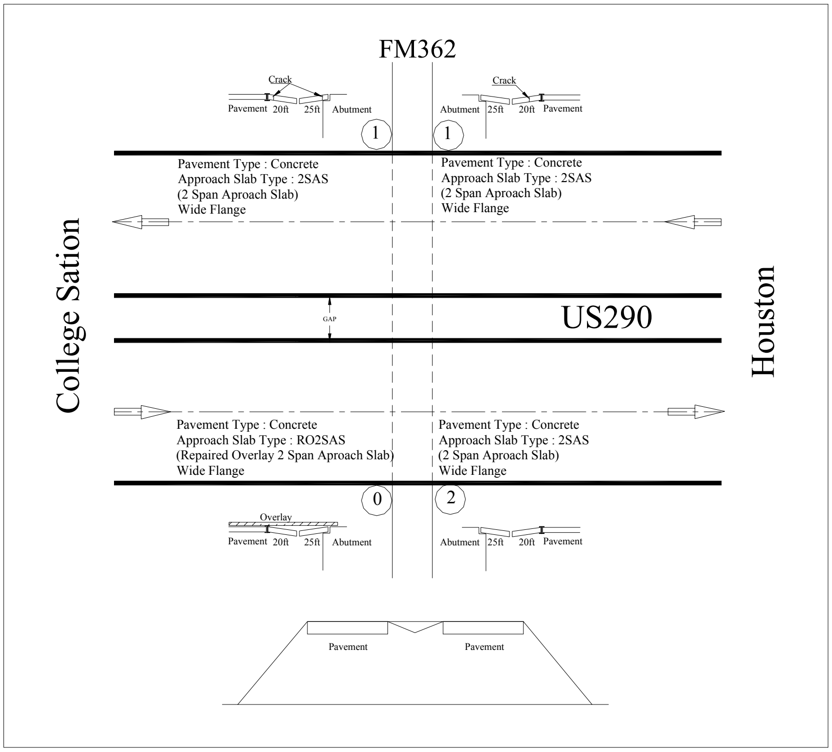


Figure 2.3. Site Description of US290 over FM362.

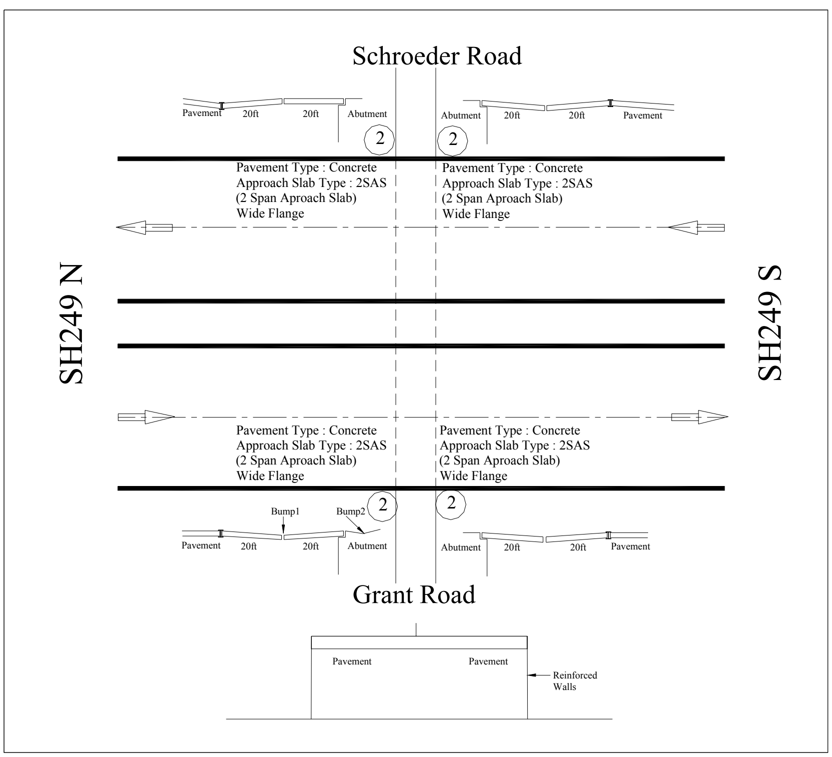
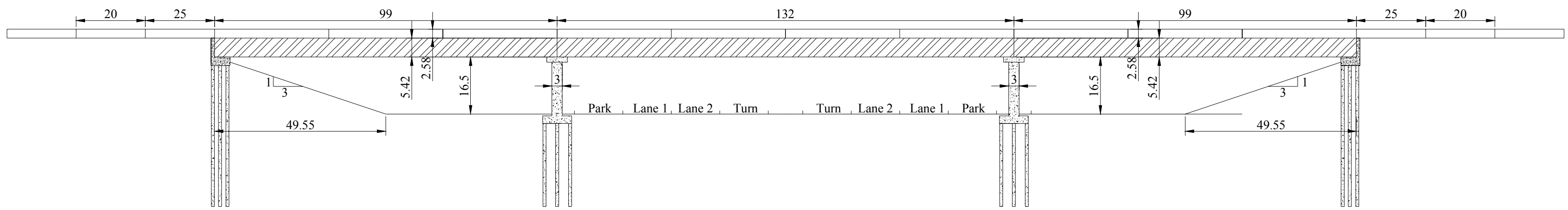
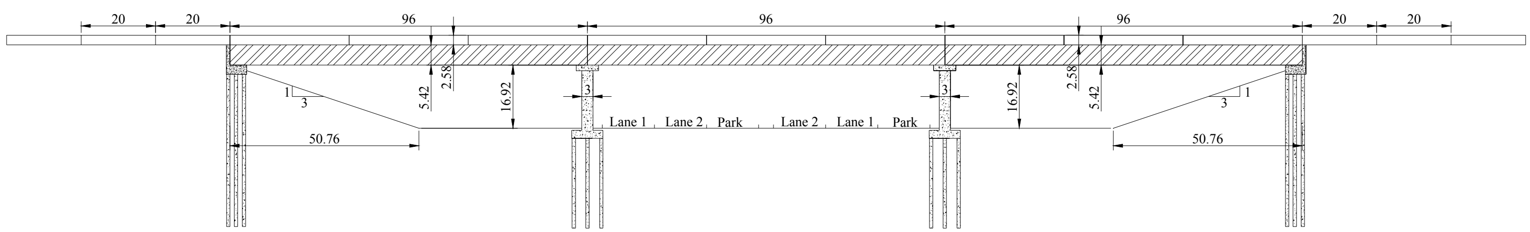


Figure 2.4. Site Description of SH249 at Grant Rd.



US290 over FM362

Unit: ft



SH249 at Grant Rd

Figure 2.5. Cross Sections of the Test Sites.

Field and Laboratory Tests

Researchers performed several field and laboratory tests. Ground penetration radar tests, continuous shelby tube sampling, cone penetration tests (CPTs), and Geogauge tests were performed in the field. Water content tests, unit weight tests, atterberg limit tests, sieve analyses, triaxial tests, compaction tests, and Geogauge tests were also done in the laboratory. Typical results are presented in Figures 2.6 to 2.10 and Tables 2.4 and 2.5. The profilometer elevation profiles and the acceleration profiles obtained by double differentiation of the elevation profiles are shown in Figures 2.11 to 2.14. The first year report gives the detailed test results.

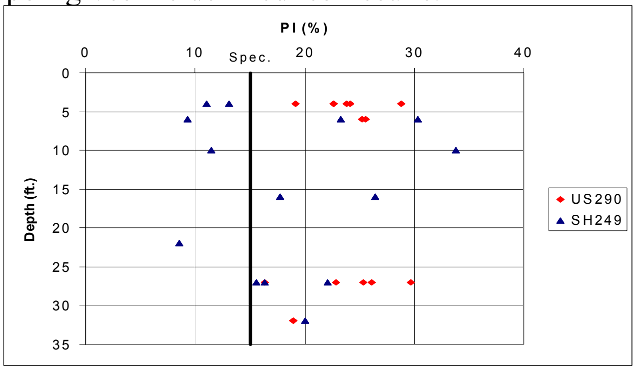


Figure 2.6. Atterberg Limit Test Results.

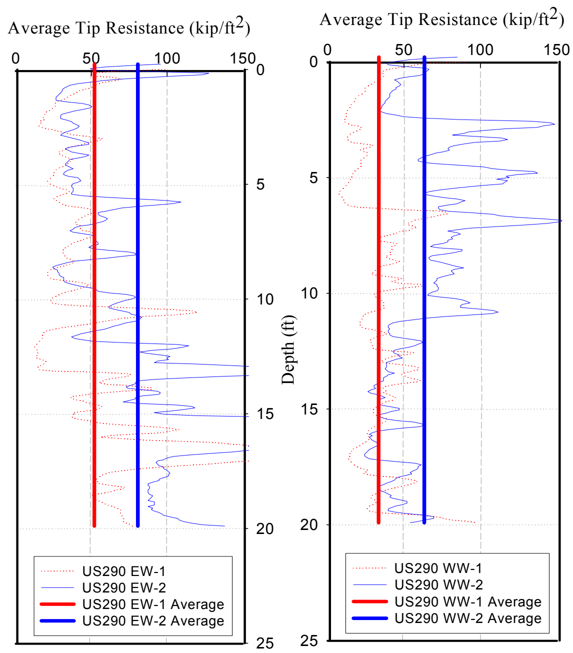


Figure 2.7. CPT Results of US290 over FM362.

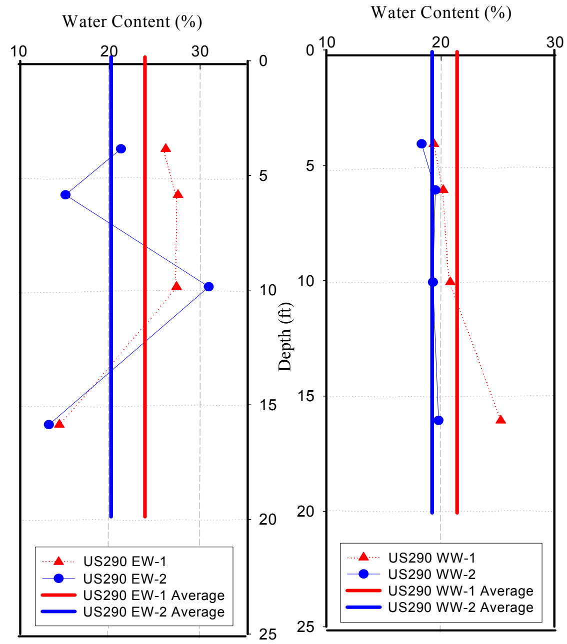


Figure 2.8. Water Content Test Results of US290 over FM362.

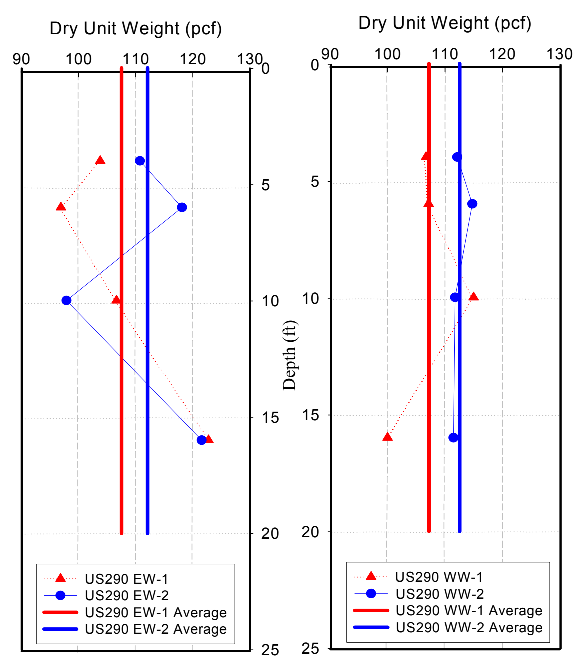


Figure 2.9. Unit Weight Test Results of US290 over FM362.

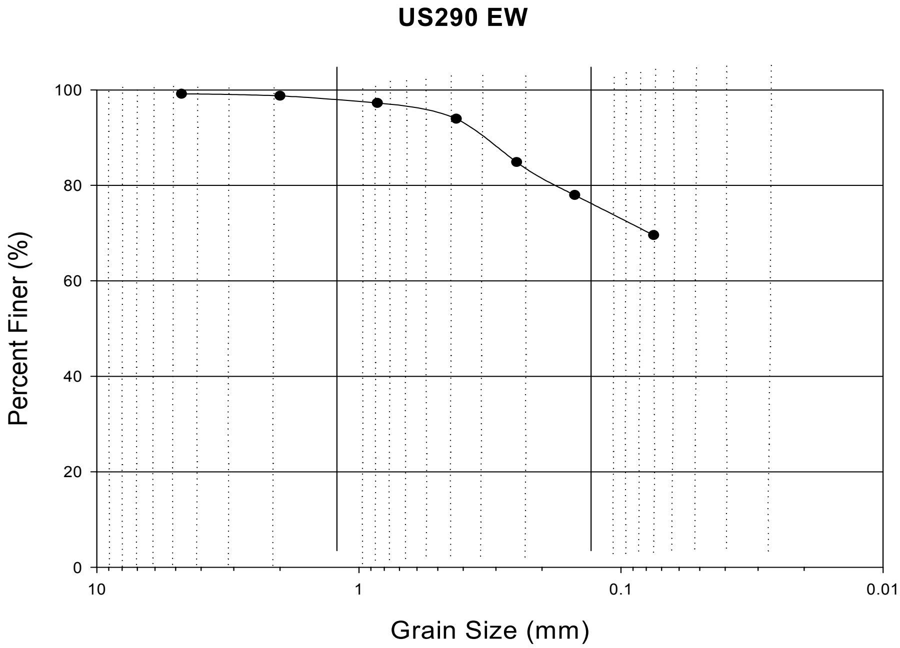


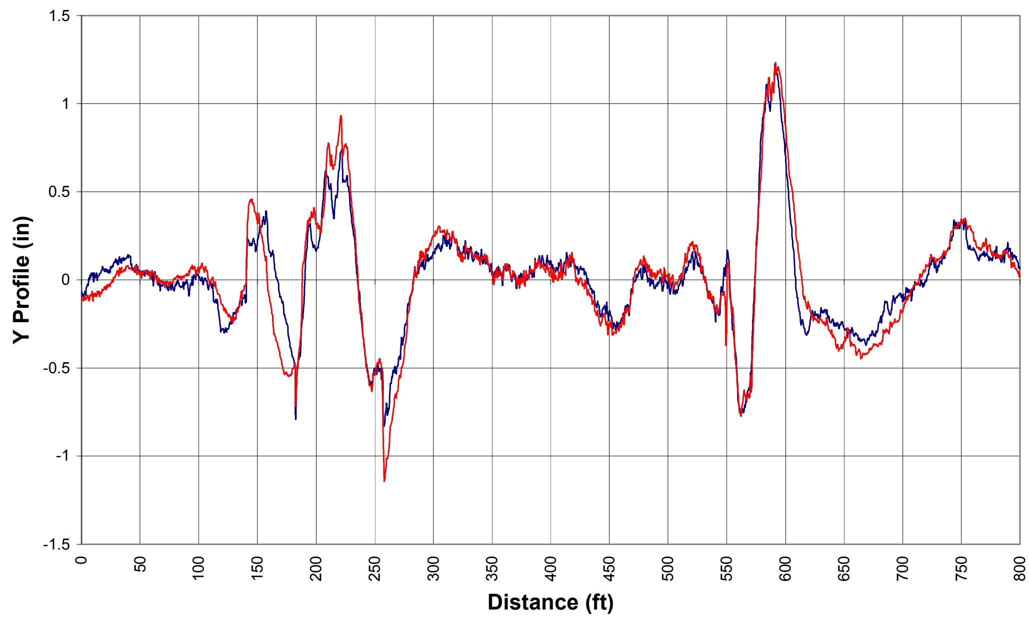
Figure 2.10. Sieve Analysis Test Results of US290 over FM362.

Table 2.4. Dry Density Test Results.

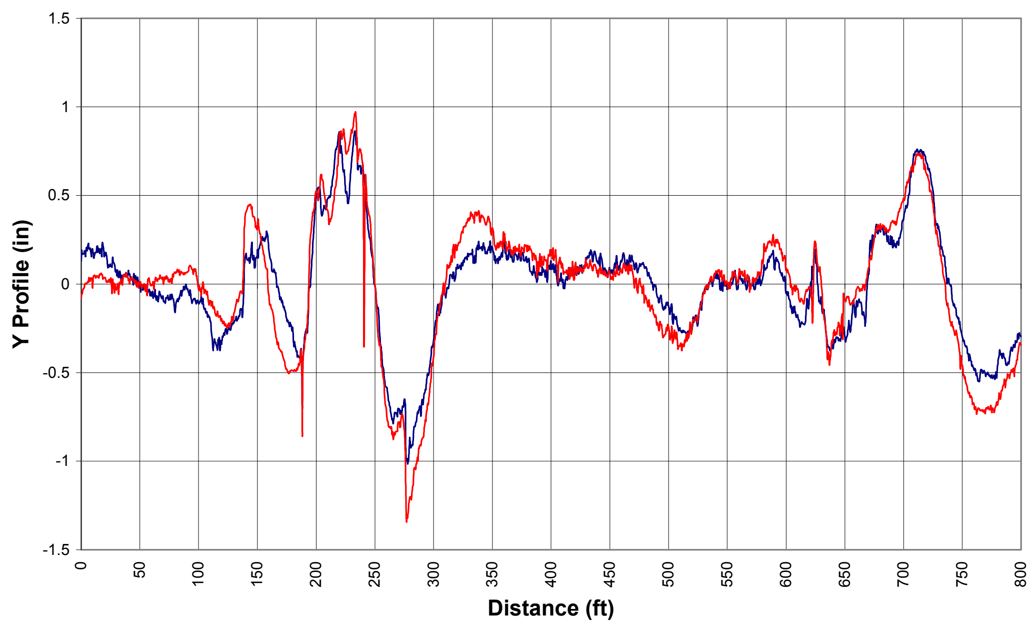
Test Site	Field Dry Density (pcf)	Lab. Max. Dry Density (pcf)
US290 WE	108.45 (94%)	114.86
US290 WW	109.45 (98%)	111.29
SH249 NS	91.44 (77%)	117.90
SH249 SS	107.43 (95%)	113.65

Table 2.5. Summary of Bump Indices.

Method	Site Rating	Remark
Visual Inspection	1-2	Slope > 1/200
IRI	820	A Rough Unpaved Road Condition
PSR	0	Really Poor Condition

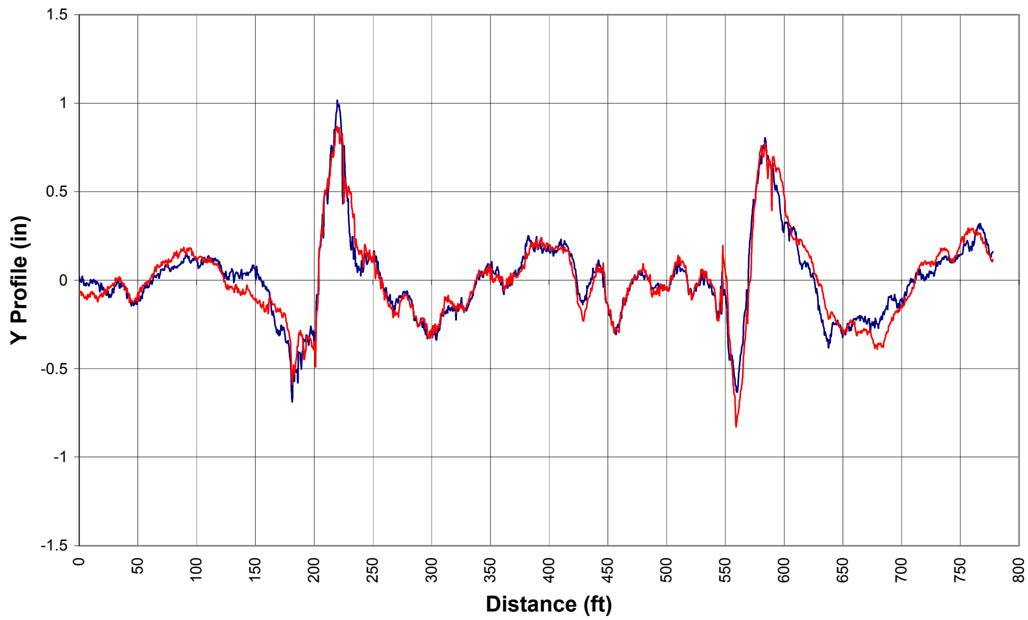


(a) Profile of US290 Eastbound Measured on April 6, 2001

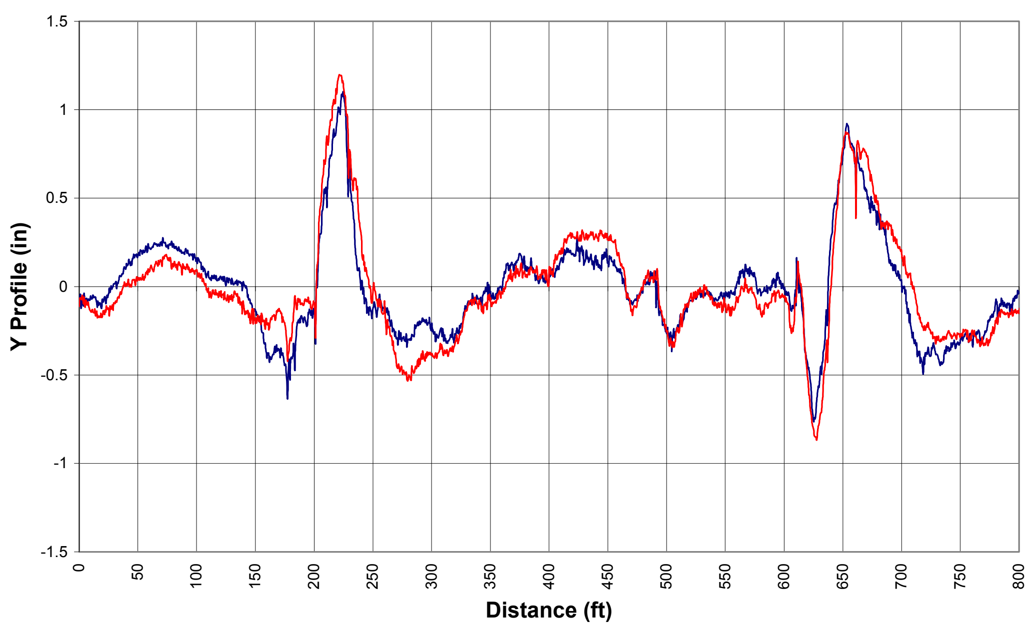


(b) Profile of US290 Eastbound Measured on March 18, 2002

Figure 2.11. Profilometer Test Results of US290.

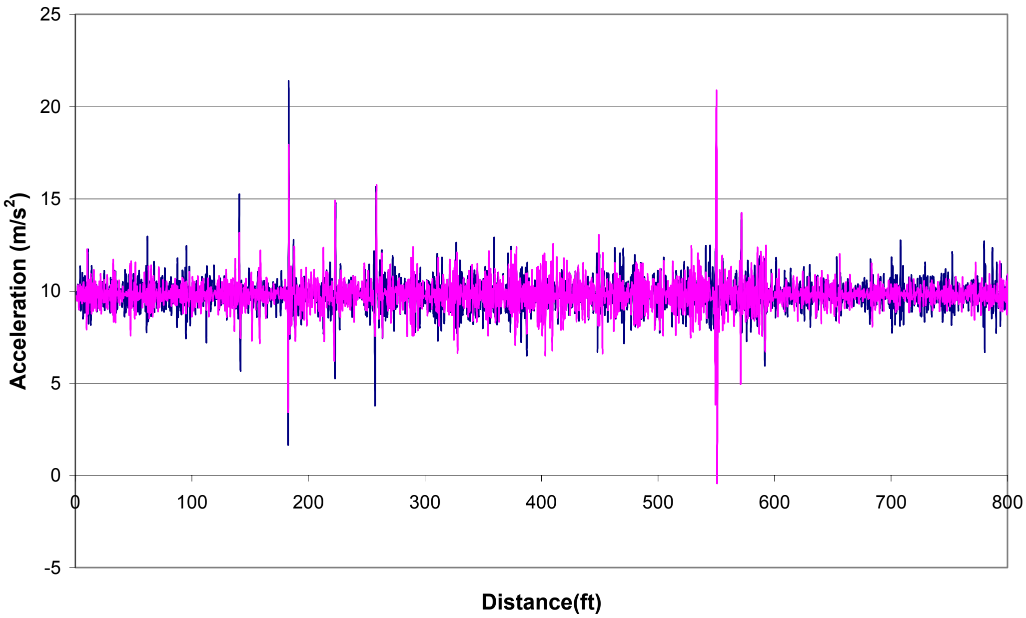


(c) Profile of US290 Westbound Measured on April 6, 2001

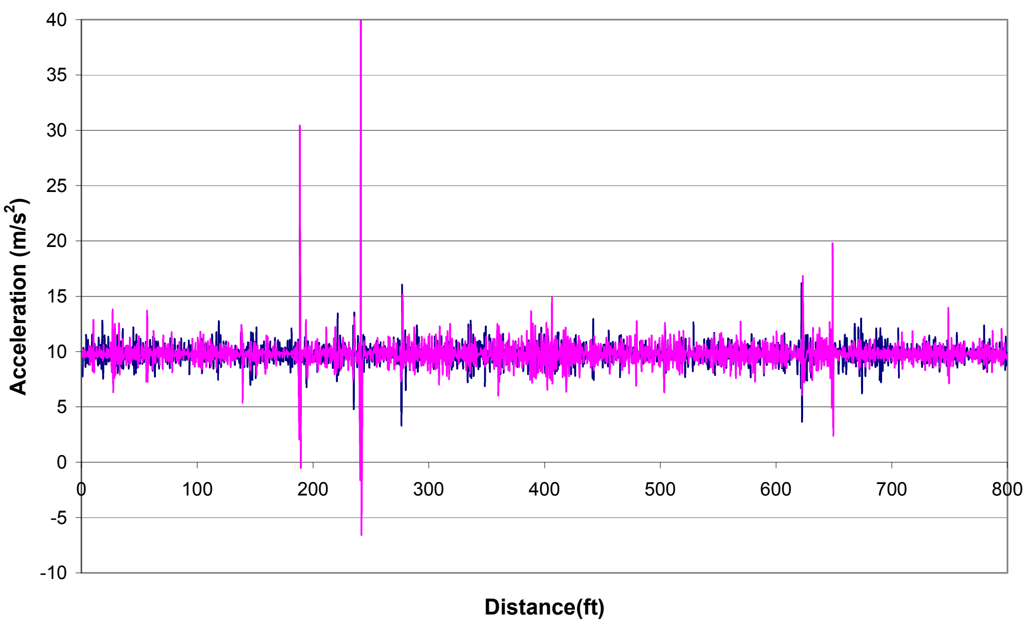


(d) Profile of US290 Westbound Measured on March 18, 2002

Figure 2.11. Profilometer Test Results of US290 (cont.).

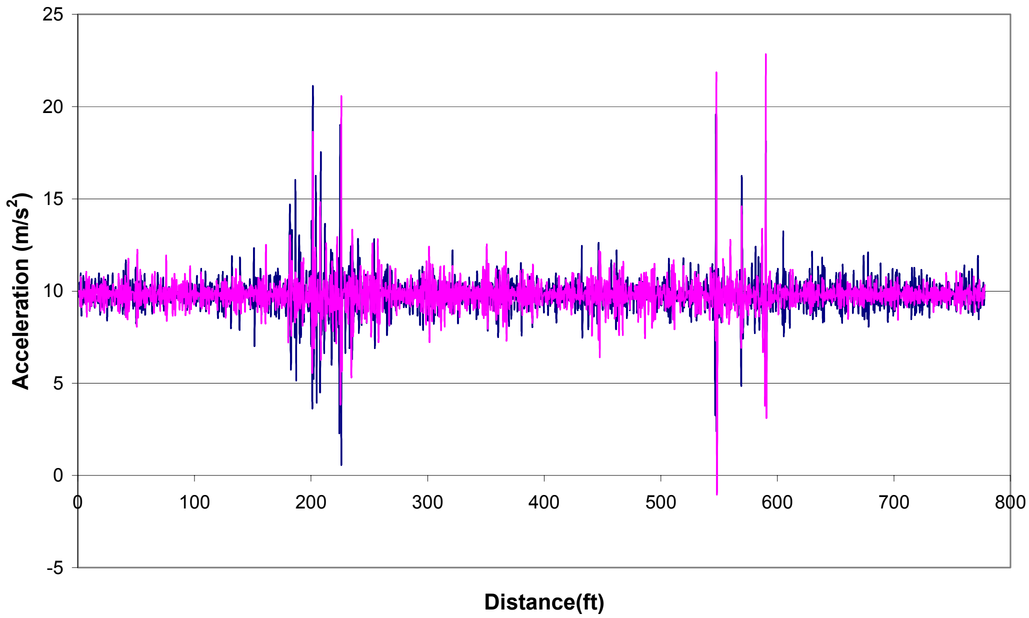


(a) Acceleration Calculated from Profile of US290 Eastbound Measured on April 6, 2001

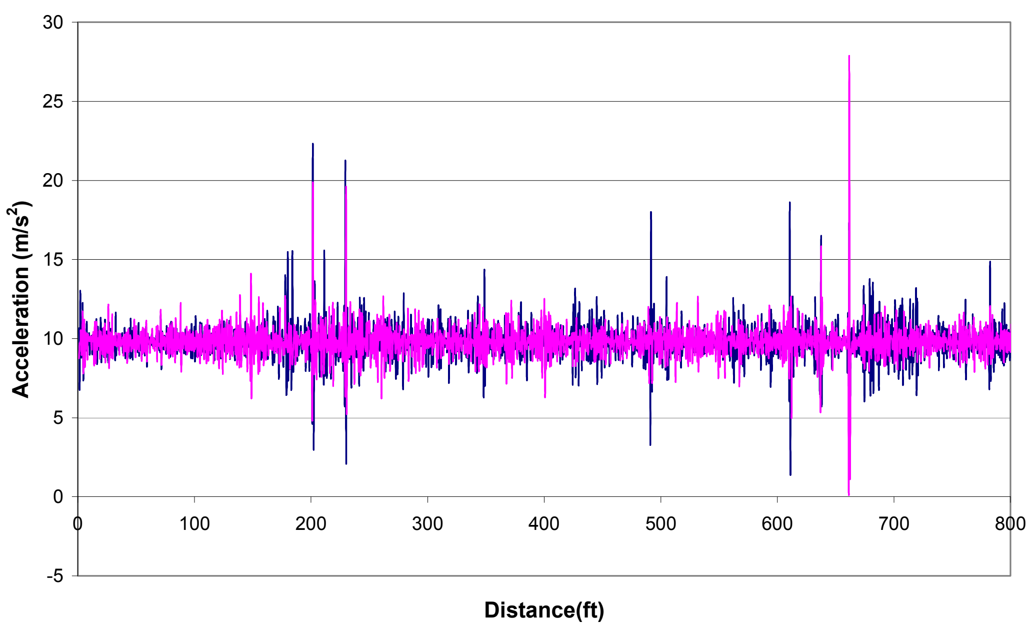


(c) Acceleration Calculated from Profile of US290 Eastbound Measured on March 18, 2002

Figure 2.12. Calculated Accelerations at US290.

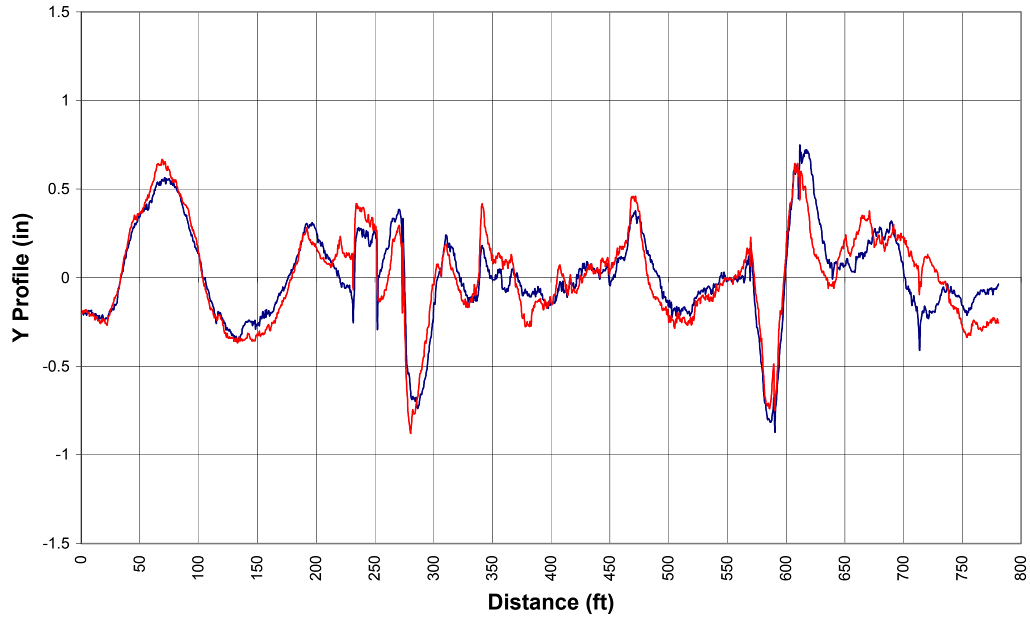


(c) Acceleration Calculated from Profile of US290 Westbound Measured on April 6, 2001

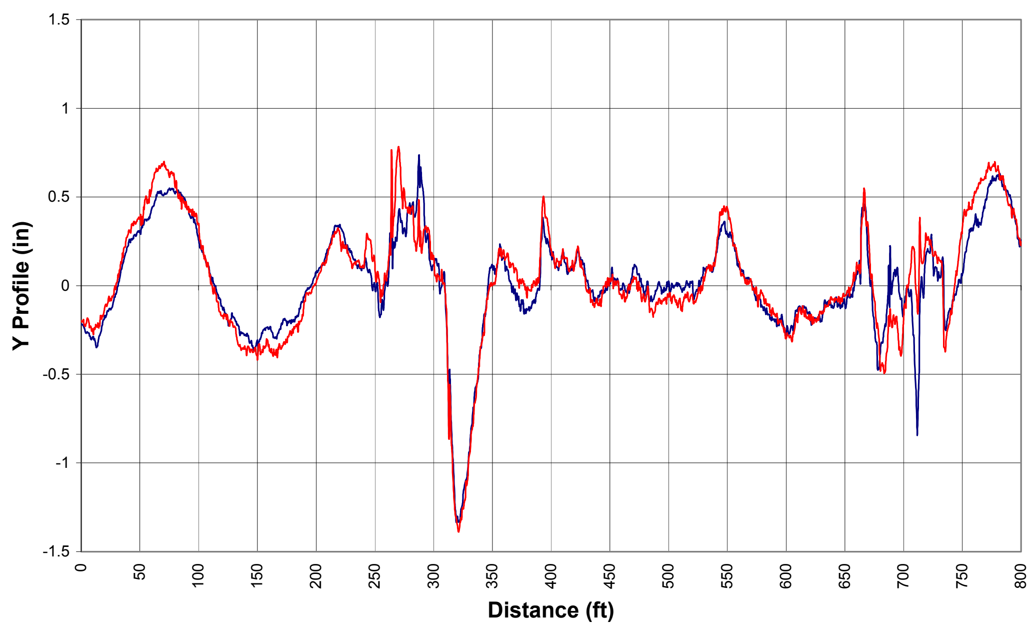


(b) Acceleration Calculated from Profile of US290 Westbound Measured on March 18, 2002

Figure 2.12. Calculated Accelerations at US290 (cont.).

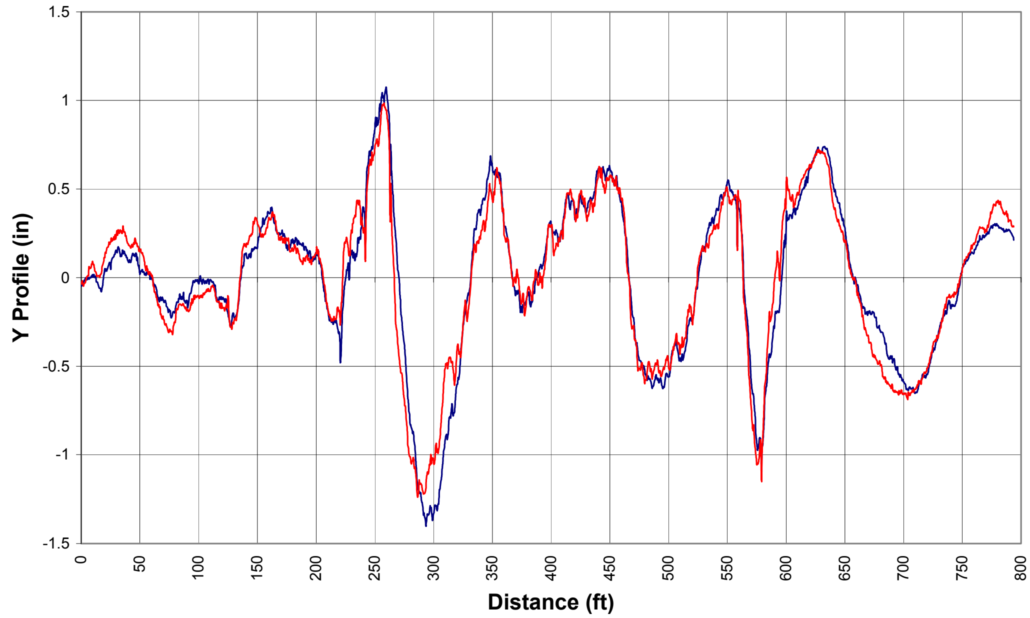


(a) Profile of SH249 Southbound Measured on April 6, 2001

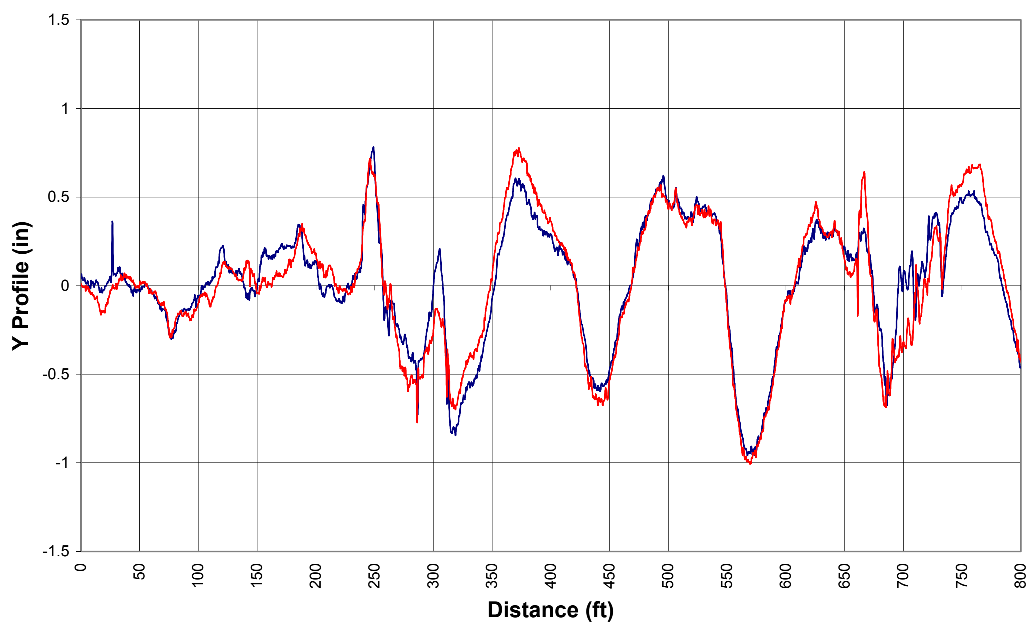


(b) Profile of SH249 Southebound Measured on March 18, 2002

Figure 2.13. Profilometer Test Results of SH249.

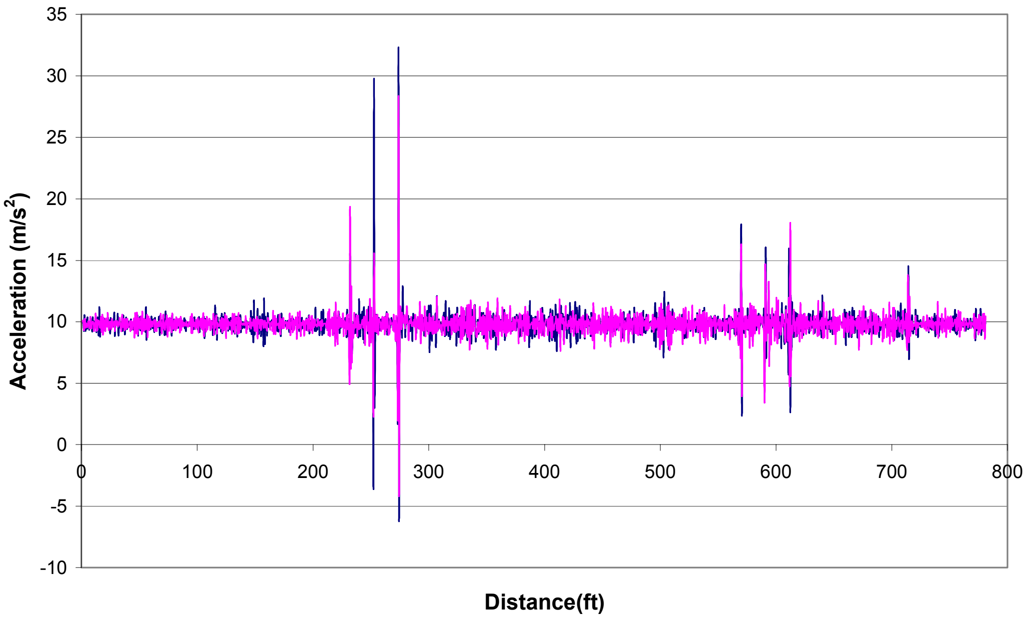


(c) Profile of SH249 Northbound Measured on April 6, 2001

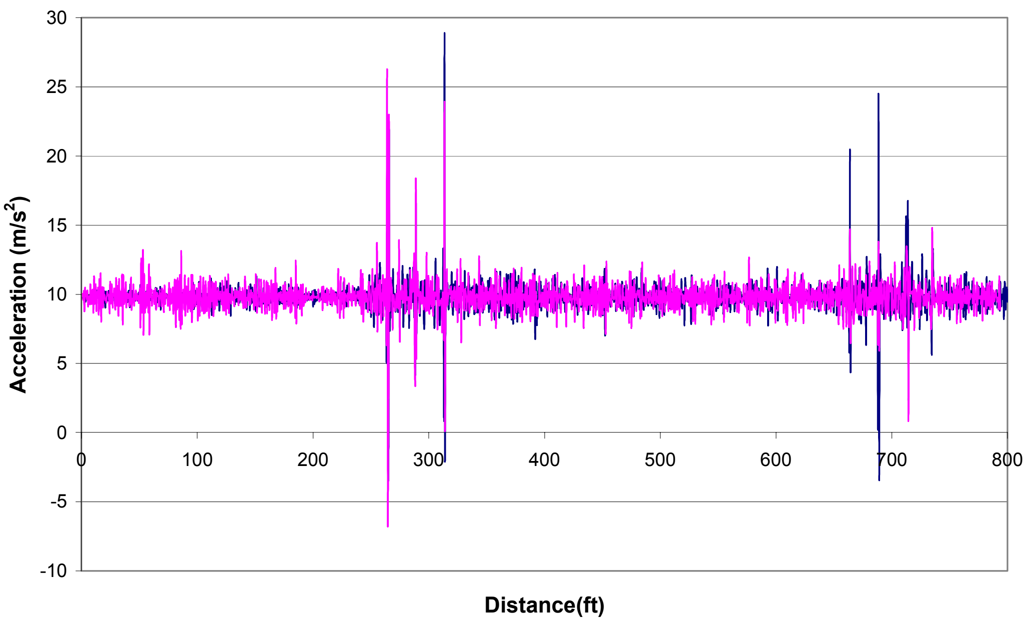


(d) Profile of SH249 Northbound Measured on March 18, 2002

Figure 2.13. Profilometer Test Results of SH249 (cont.).

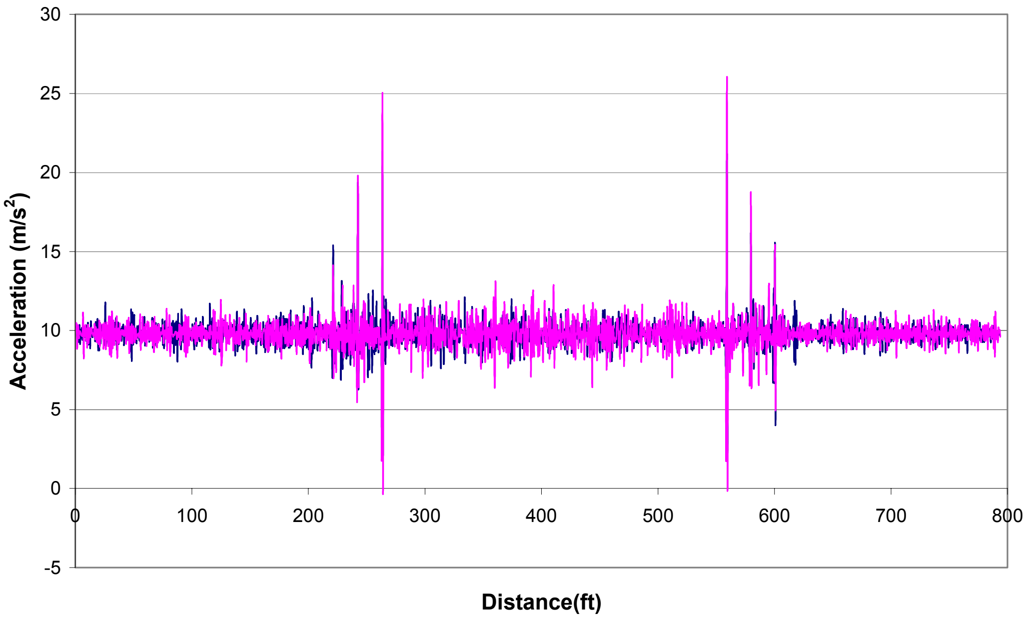


(a) Acceleration Calculated from Profile of SH249 Southbound Measured on April 6, 2001

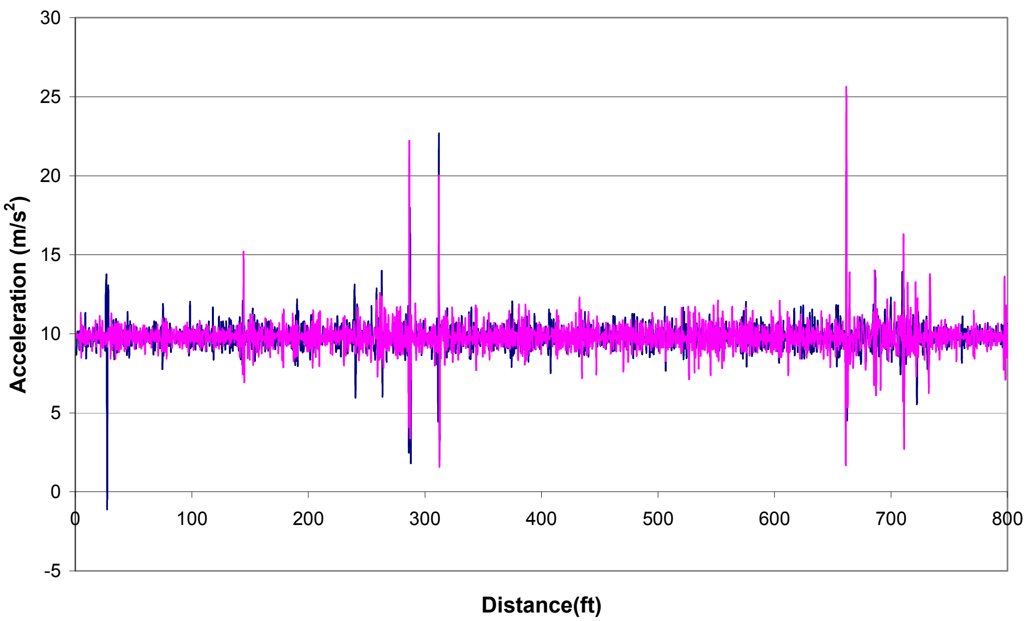


(b) Acceleration Calculated from Profile of SH249 Southbound Measured on March 18, 2002

Figure 2.14. Calculated Accelerations at SH249.



(c) Acceleration Calculated from Profile of SH249 Northbound Measured on April 6, 2001



(d) Acceleration Calculated from Profile of SH249 Northbound Measured on March 18, 2002

Figure 2.14. Calculated Accelerations at SH249 (cont.).

Conclusions

1. The profilometer gave bump amplitudes varying from 1.15 to 2.35 inches on April 2001 and from 0.76 to 2.12 inches on March 2002; transition slopes as steep as 1/100; international roughness indices (IRI) as high as 820, indicating a rough unpaved road condition; and present serviceability indices of 0, indicating really poor condition.
2. The profilometer test performed one year after the first one indicated that some of the bumps had decreased and some had stayed the same, while others had increased. Therefore, bumps are dynamic features that may be tied to the weather through the shrink-swell nature of some soils used for embankment fills.
3. The vertical acceleration of the car-wheel obtained by double differentiation of the elevation profile was up to 40 m/sec^2 or 4 g's at the bump location.
4. Close to the bridge abutment, the cone penetrometer (CPT) resistance was 33.8 percent lower on the average and the water content was 10.5 percent higher on the average than the values away from the abutment.
5. The compaction level within the embankment below the bump averaged 96 percent of the Standard Proctor maximum dry unit weight.
6. The soil of the embankment fill had a PI varying from 8.52 to 33.77 with an average equal to 20.96.
7. The ground penetrating radar indicated that there were no voids under the pavement.

CHAPTER 3:
CURRENT PRACTICE

Many components are involved in the development of the bump at the end of the bridge, and many factors contribute to its existence. Briaud et al. (1997) identified those components and the factors contributing to its existence (Figure 3.1). To understand those components and factors, current U.S. practices for the connection between the bridge and the embankment including approach slabs are reviewed in this chapter. This chapter consists of three sections. The first section covers planning, design, and construction practices. The second section describes the existing maintenance methods for approach slabs. The current practice in Houston, Texas is discussed in the third section.

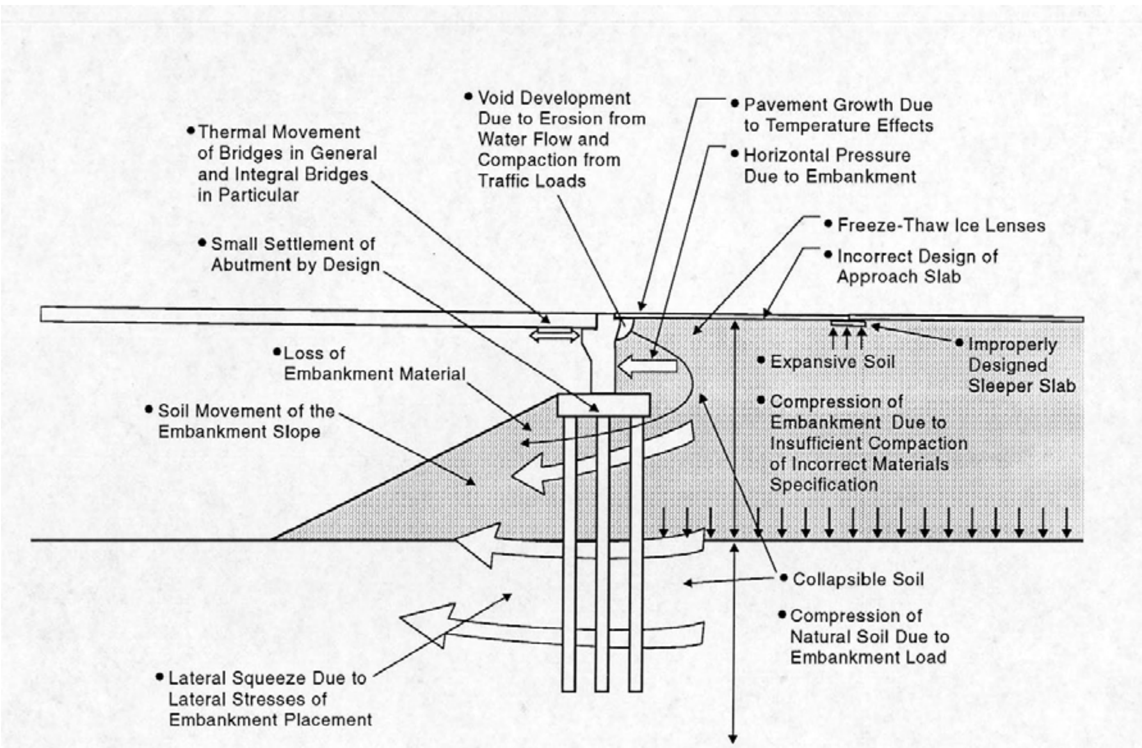


Figure 3.1. Problems Leading to the Existence of a Bump (after Briaud, 1997).

Planning, Design, and Construction Practices

Geotechnical Investigations

Geotechnical investigations are integrated in the investigation for the bridge structure. The American Association of State Highway and Transportation (AASHTO) manual (1984) on subsurface investigation and the TxDOT Geotechnical manual (2000) present the guidelines and very comprehensive information on methodology for subsurface investigations. This investigation provides information on the depth, thickness, and classification of all soil strata. The AASHTO subsurface investigation manual (1984) also presents suggested guidelines for the spacing and depth of borings for structures and embankments. For embankments higher than 15 ft, the recommended boring spacing is a maximum of 200 ft, with the interval decreased to 100 ft when erratic conditions or compressible soils are encountered. For each bridge abutment, a maximum of two borings is recommended, and additional borings are suggested when the abutment exceeds 100 ft in length or has wingwalls more than 20 ft long. The recommended depth of borings is the depth at which the net stress increase caused by imposed foundation loads is less than 10 percent of the effective overburden pressure at that depth, unless rock or dense soil known to lie on rock is encountered above that depth (Wahls, 1990).

Bridges

Two major design concepts, conventional bridges and integral abutment bridges, are currently used for road bridges. The conventional design type has a superstructure resting on an abutment at each end as shown in Figure 3.2. The basic concept of this design is to make the superstructure unconstrained. Expansion joints and bearings at each end of the superstructure are used to accommodate the seasonal relative movement between superstructure and abutment and to prevent temperature-induced stress from developing within the superstructure. Conventional bridges have shown good performance for a long time, but they lead to a high maintenance cost because of the corrosion and other physical deterioration of the bridge bearings and joints.

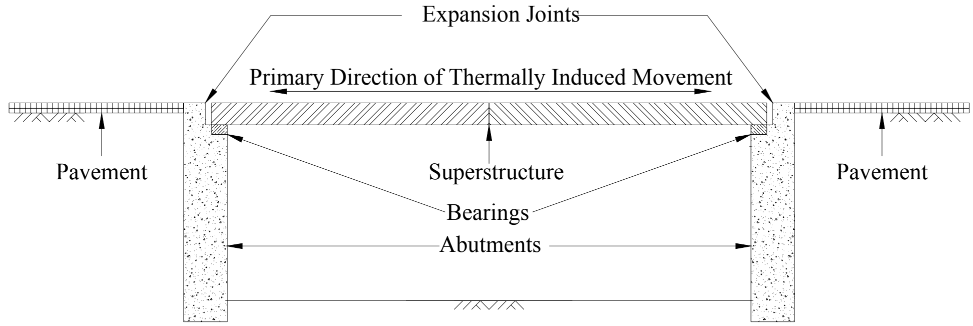


Figure 3.2. Traditional Design Concept (after Horvath, 2000).

Because of these flaws, a new design concept consists of physically and structurally connecting the superstructure and abutments as shown in Figure 3.3. This type of bridge usually has an approach slab to provide a smooth transition between the integral abutment bridge (IAB) and adjacent approach embankments. In doing so, some problems associated with the conventional bridge concept can be minimized but other problems such as the bump at the end of the bridge can be exacerbated. Horvath (2000) described that in this scenario the root cause of problems has shifted from being primarily structural to being primarily geotechnical in nature.

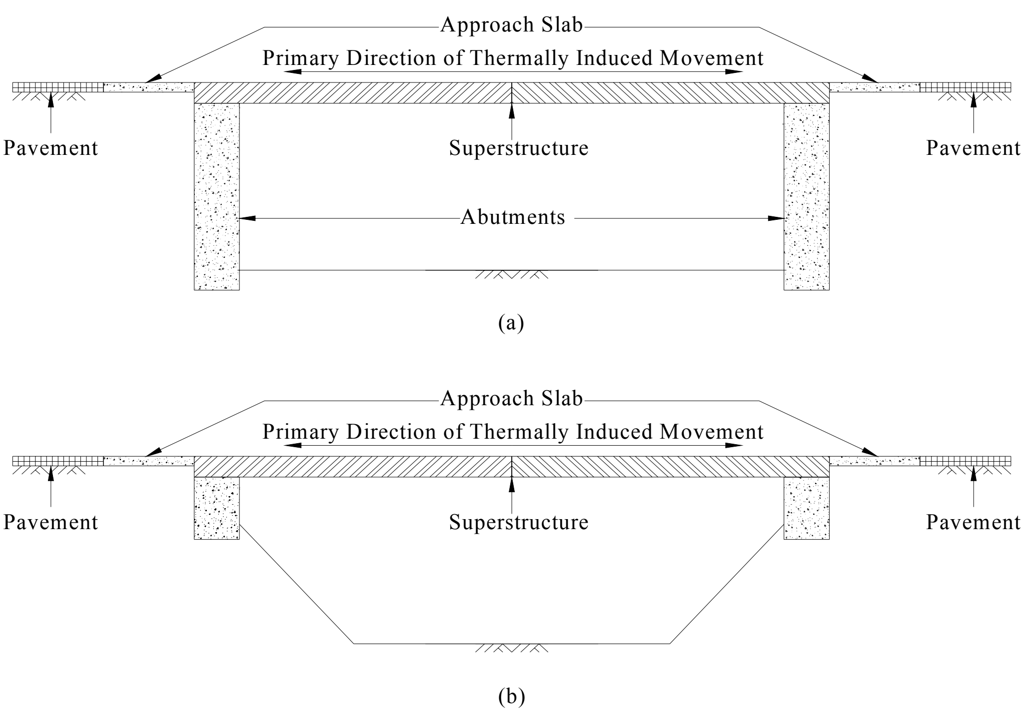


Figure 3.3. Integral Abutment Bridge Design Concept (after Horvath, 2000).

Approach Embankments

Most bridge approach embankments are constructed by conventional rolled earth procedures, and there are many types of approach fill materials that can be used. Fill material that is readily available may be more economical but may not perform as well as a select fill material. For this reason, some states specify select materials and increased compaction requirements, especially near the abutment. For example, California specifies fill with a maximum Plastic Index (PI) of 15 and fewer than 40 percent fines within 150 ft of an abutment wall (Figure 3.4), and the required relative compaction is increased to 95 percent from 90 percent within this zone. The approach embankment typically is compacted in 6 to 24-inch layers, depending on the type of soil and compaction of clean granular fills, and even for such soils thin lifts must be used adjacent to the abutment (Wahls, 1990).

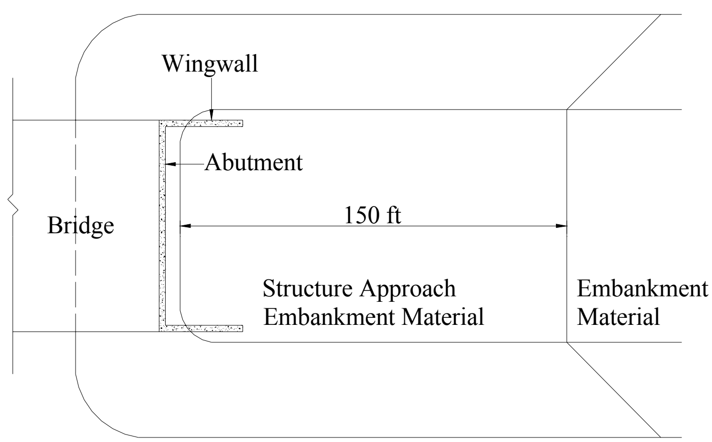


Figure 3.4. Limits of Structure Approach Embankment Material (after Caltrans).

Transportation Research Board (TRB) syntheses (TRB, 1969, and TRB, 1971) presented the placement procedures and compaction requirements for construction of rolled earth embankments in the late 1960s. Most agencies still require 90 to 95 percent of the maximum dry density achieved in the AASHTO T 99 Compaction Test for roadway embankments and 95 to 100 percent for bridge approaches without changing these procedures in the past 20 years (Wahls, 1990). The use of well-graded backfill with less than 5 percent finer than the 75 μm (No. 200) sieve is ideal and is strongly recommended.

Even with proper compaction, fills with significant clay content may develop time-dependent movements. Lightweight fills have been used to prevent the movements. Wahls (1990) and Elias and Christopher (1996) described the lightweight fills that have been used.

Abutments

Bridge abutments support the structural loads and are subjected to lateral earth pressures from the approach embankments. There are five types of abutment in use: 1) closed or high abutment, 2) stub or perched abutment, 3) pedestal or spill-through abutment, 4) integral abutment, and 5) mechanically stabilized abutment.

Closed abutments (Figure 3.5) have a full-height wall and wingwalls on each side. These abutments can decrease the required span length of the bridge but they must be constructed before the adjacent embankment. Therefore, it is difficult to place and compact the embankment fills at the confined space. Closed abutments are also subjected to higher lateral earth pressure than other abutment types.

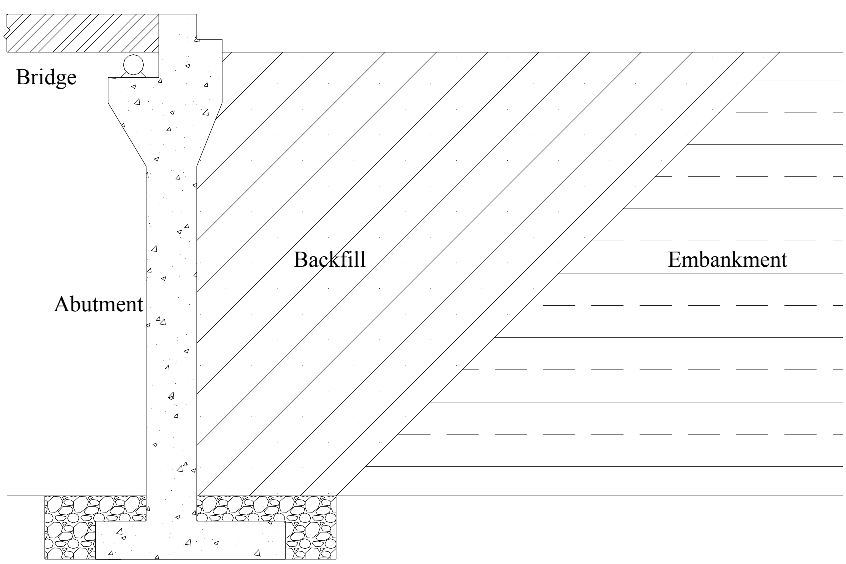
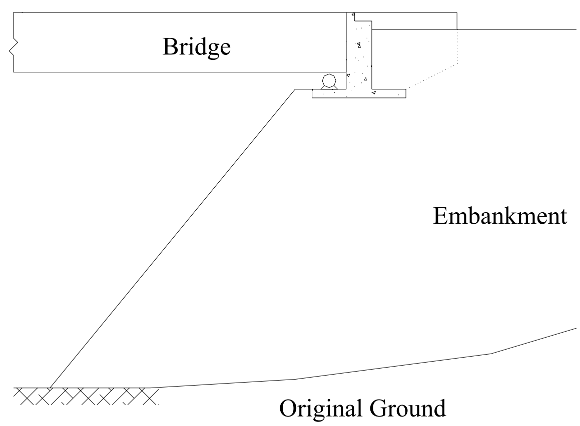
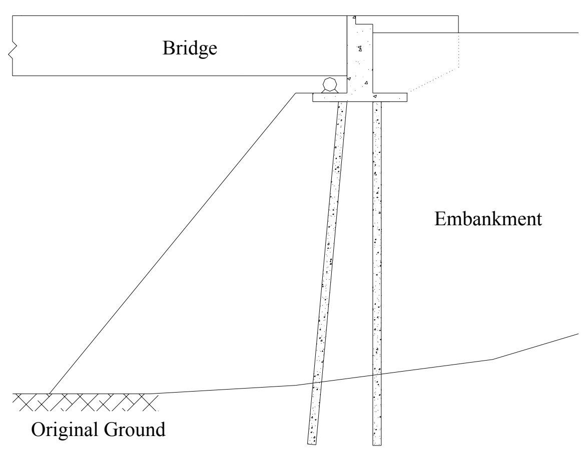


Figure 3.5. Typical Full-Height Closed Abutment (after Wahls, 1990).

Stub or perched abutments (Figure 3.6) are relatively short abutments supported on a shallow foundation in the embankment or on piles. Because stub or perched abutments are constructed in the upper part of the fill after the embankment has been completed, the lateral earth pressure is relatively small. This type of abutment is most common in Texas (Figure 3.6(b)).



(a) Spread Footing



(b) Piles

Figure 3.6. Typical Stub or Perched Abutment (after Wahls, 1990).

Pedestal or spill-through abutments, which must be constructed before the embankments, are stub abutments supported on columns, as seen in Figure 3.7. This type of abutment gets lower lateral earth pressures than closed abutments but the compaction around the pedestal is difficult. Compared to full-height closed abutments, perched abutments generally lead to smaller continuing lateral movement after construction.

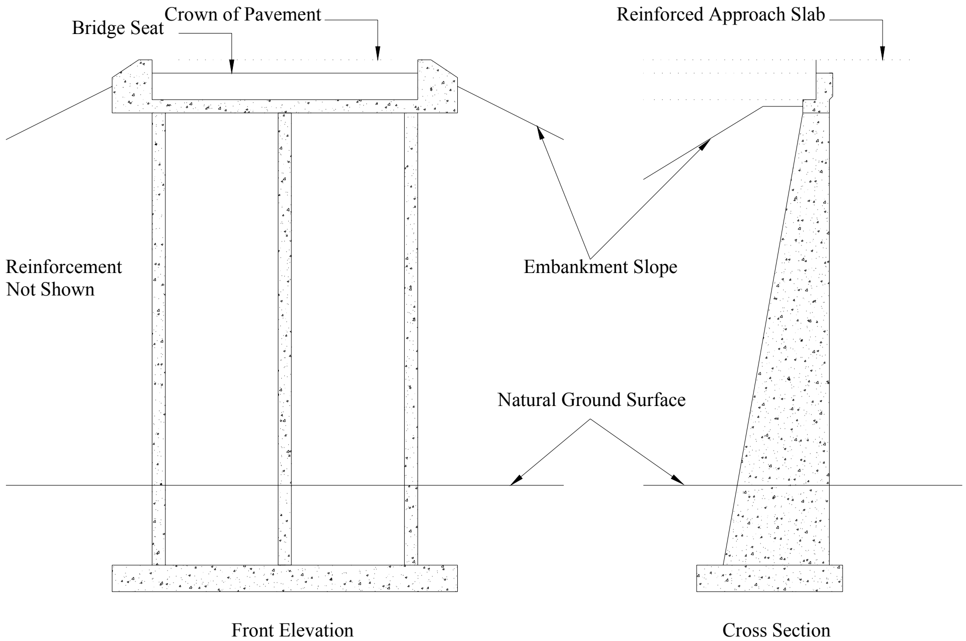


Figure 3.7. Typical Pedestal or Spill Through Abutment (after Wahls, 1990).

Integral abutments (Figure 3.8) are very similar to pedestal or spill-through abutments except that the end bend is connected to the superstructure without expansion joints. The basic concept of this abutment is to fully transfer the stress caused by thermal effect to the abutment. It can save construction and maintenance costs by eliminating expansion joints and bearing systems.

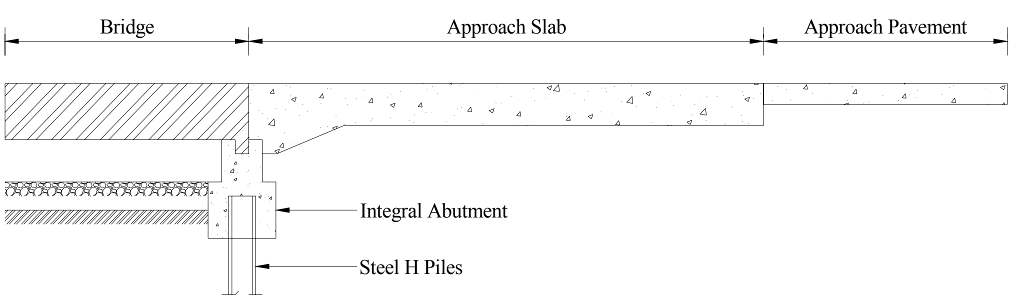


Figure 3.8. Typical Integral Abutment (after Wahls, 1990).

Mechanically stabilized abutments are stub or perched abutments founded on a spread footing resting on the reinforced embankment fill (Figure 3.9). The embankment fill is reinforced with geosynthetics or metallic reinforcement. This reinforcement absorbs the lateral pressures caused by the embankment fill. The construction of mechanically stabilized backfill (MSB) is simple and time-efficient.

Figure 3.9. Schematic Diagram of Mechanically Stabilized Abutment (after Wahls, 1990).

A wingwall (Figure 3.10) is usually constructed to contain the approach fill material near the abutment. It can be perpendicular to the abutment or extend out at an angle.

Bridge abutments are usually supported on bored piles, driven piles, or spread footings. The foundation type depends on the soil, the type of bridge, and environmental factors.

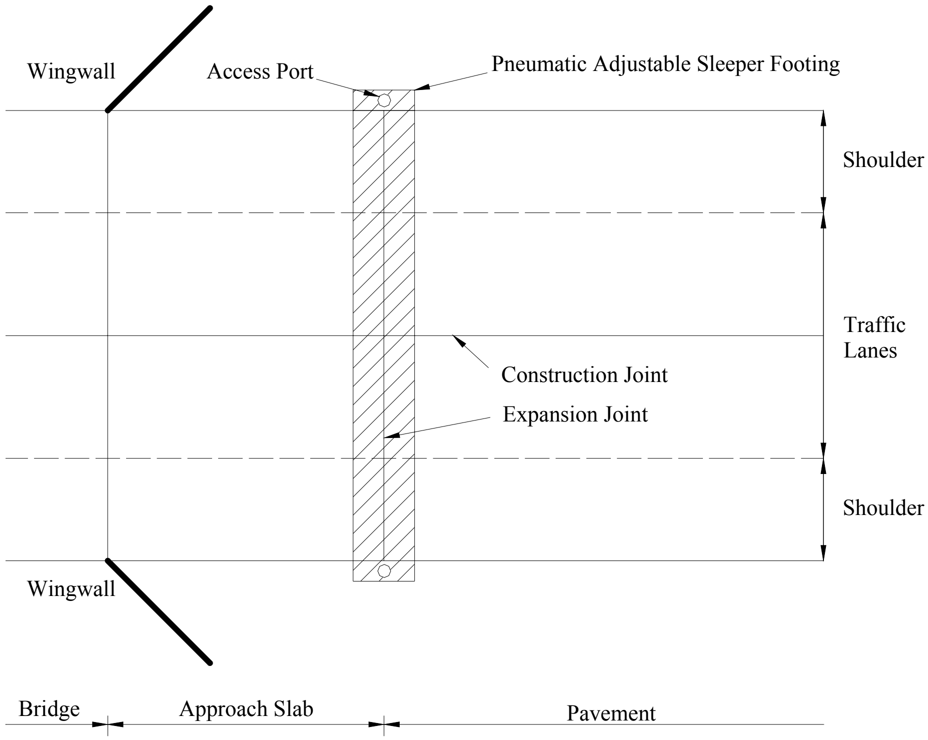


Figure 3.10. Plan View of an Approach System (after Tadros and Benak, 1989).

Approach Slabs

Approach slabs are reinforced concrete slabs used to provide a smooth transition between the bridge deck and the roadway pavement (Figure 3.11). They are used in about 80 percent of new bridges (Schaefer and Koch, 1992). Most approach slabs are 20 to 40 ft long. The thickness of approach slabs also varies. Typically they are 9 to 12 inches thick. The slab width is the same as the bridge deck. The slabs may be supported at both ends; the bridge end is supported by the abutment and the pavement end by a sleeper slab or directly by the roadway embankment. The sleeper slab is a slab that underlies and supports the ends of the approach slab and the adjacent roadway pavement. Figure 3.12 shows some typical joints at integral and non-integral abutments. Expansion joints at the roadway end of the approach slab are shown in Figures 3.11 and 3.13. A pressure-relief joint, which is used when there is an expansion joint at the abutment, is shown in Figure

3.14. Some approach slab details and questionnaire results about approach slabs (Hoppe, 1999) are presented in Appendices A and B.

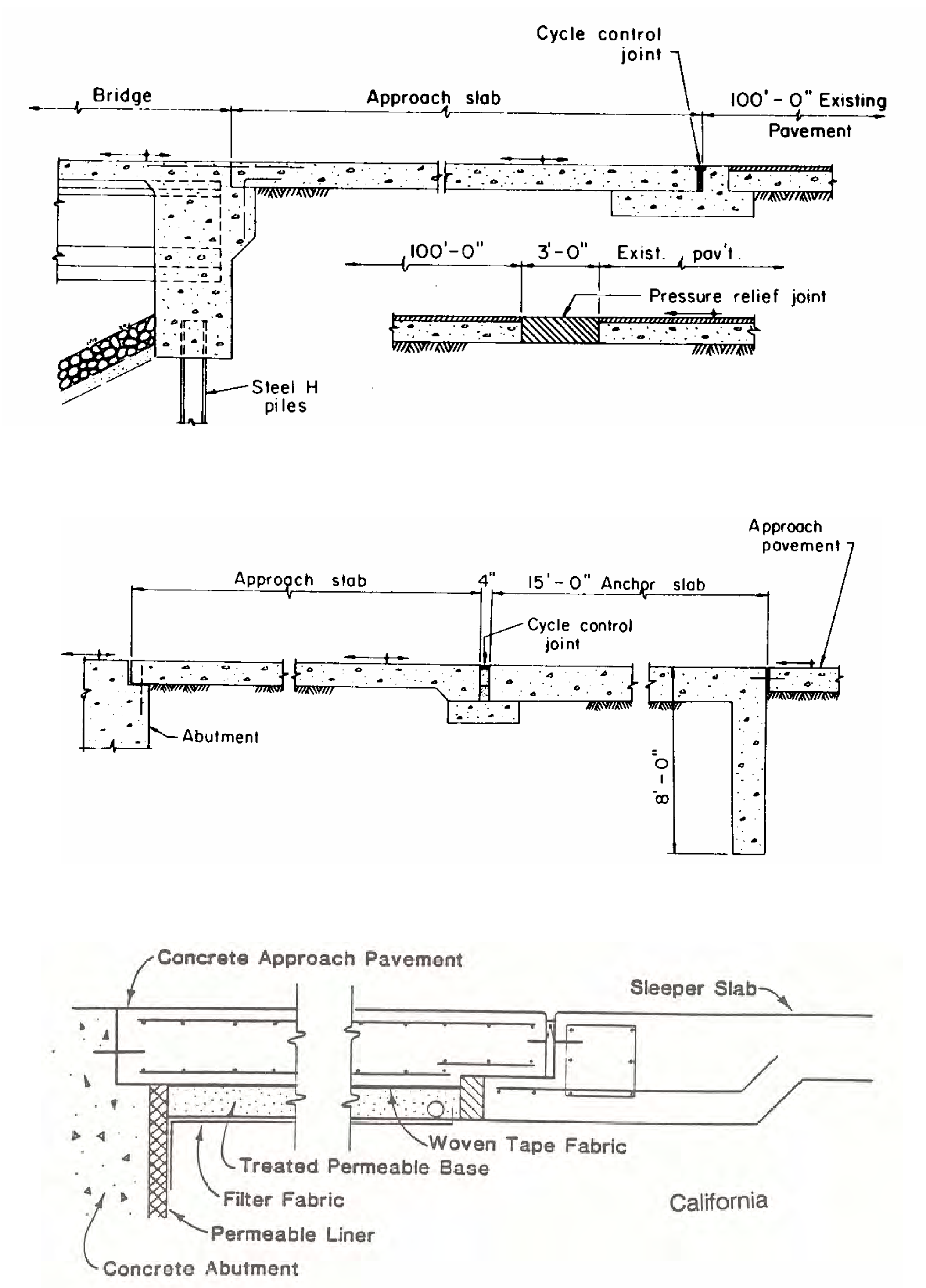


Figure 3.11. Examples of Approach Slabs (after Burke, 1987).

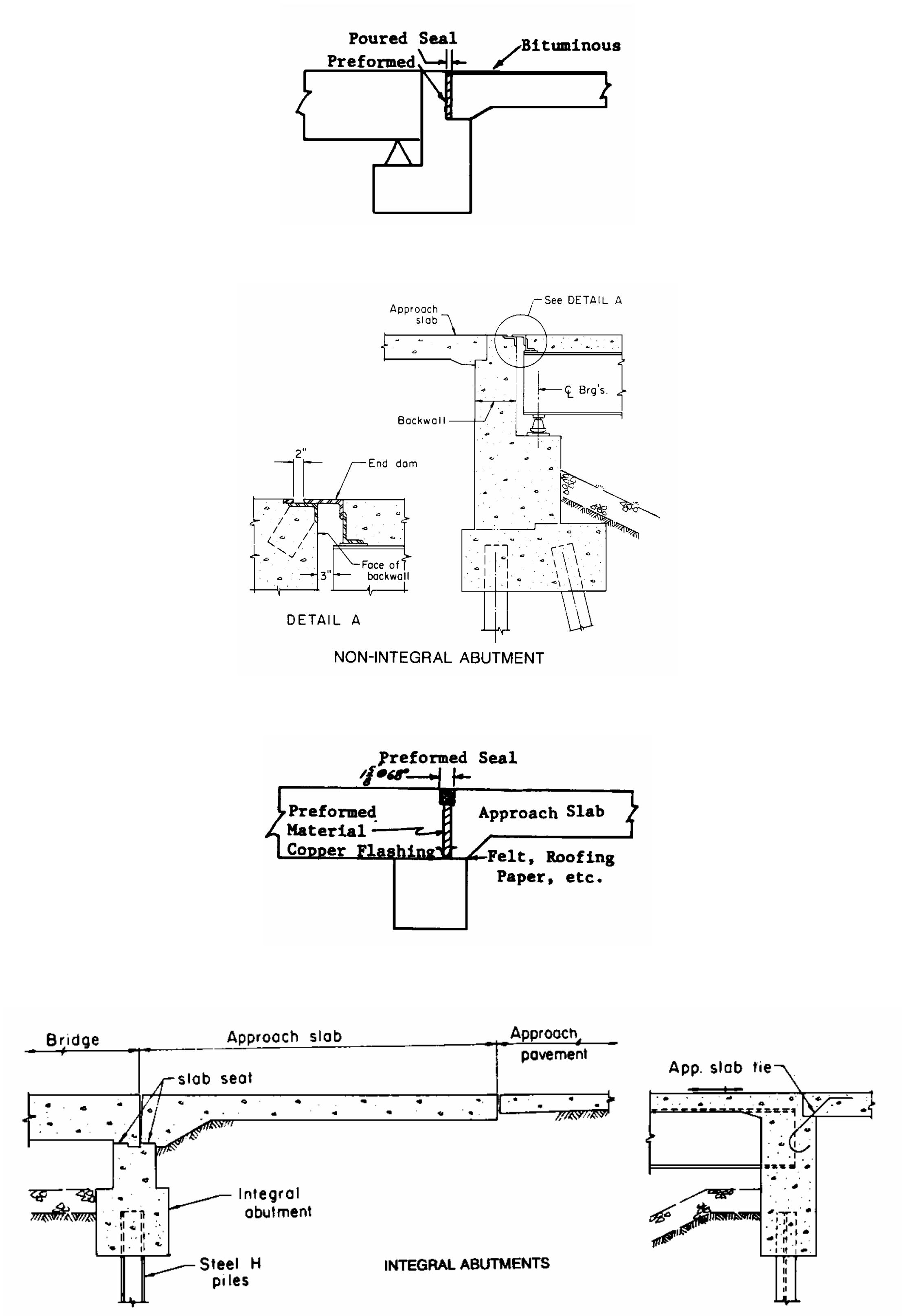


Figure 3.12. Approach Slab/Abutment Joints (after Burke, 1987).

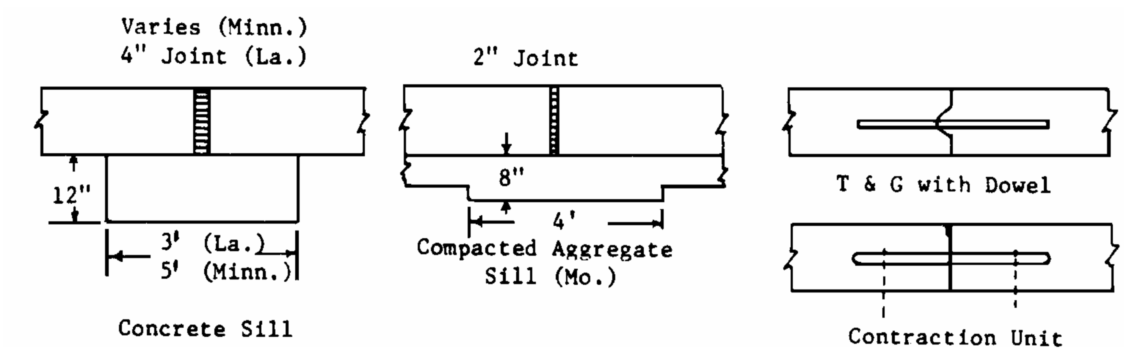


Figure 3.13. Approach Slab/Roadway Joints (after Burke, 1987).

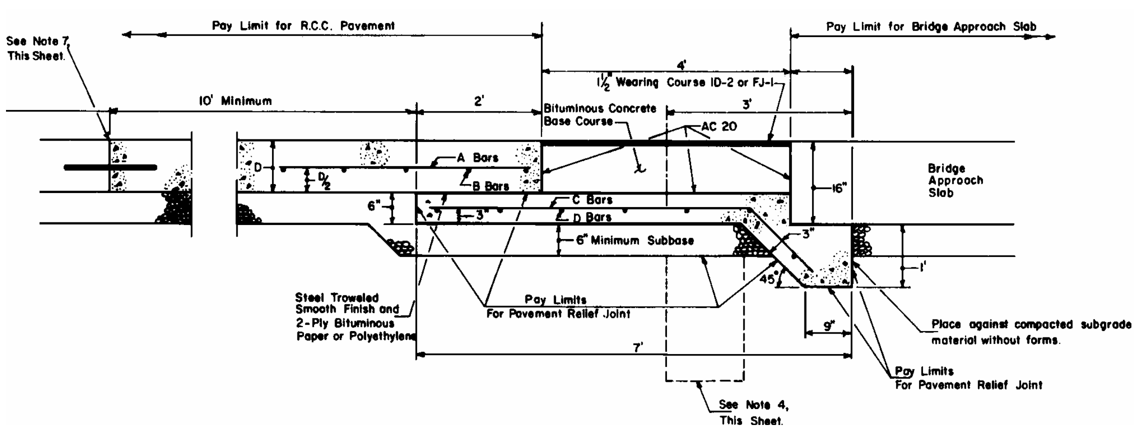


Figure 3.14. Pressure-Relief Joint (after Briaud et al., 1997).

Drainage Provisions

Both surface and subsurface drainage systems are very important at bridge approaches. The surface runoff should be routed away from the bridge/approach joint. It is essential to keep water from infiltrating the fill beneath the approach slab and behind the abutment. One recommendation for an appropriate surface drainage system is to place the wingwalls beyond the bridge end panel (Bellin, 1993). Another recommendation is to have a pavement wingwall assembly as shown in Figure 3.15 (Briaud et al., 1997).

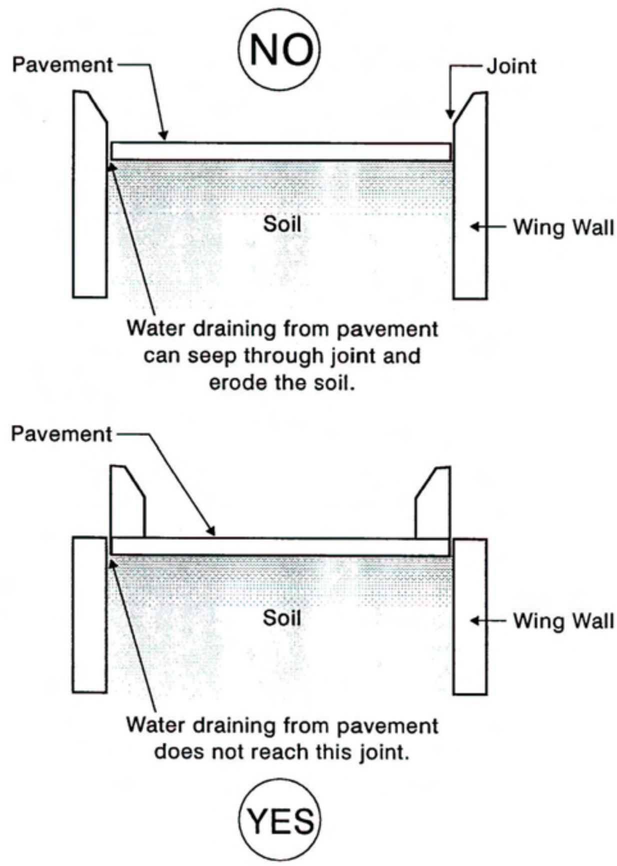


Figure 3.15. Cross Section Showing Wingwall and Drainage Detail (after Briaud et al., 1997).

Chini et al. (1993), Wahls (1990), and Stark et al. (1995) discussed bridge approach drainage. Wahls suggests the use of gutters and paved ditches to direct surface water away from the bridge approach system. Figure 3.16 shows a geocomposite drainage system, which is a prefabricated subsurface drainage system. Note that these types of drainage systems must be designed for site-specific conditions and they must be able to withstand the earth pressure (Briaud et al., 1997). Examples of bridge approach drainage details are shown in Appendix C.

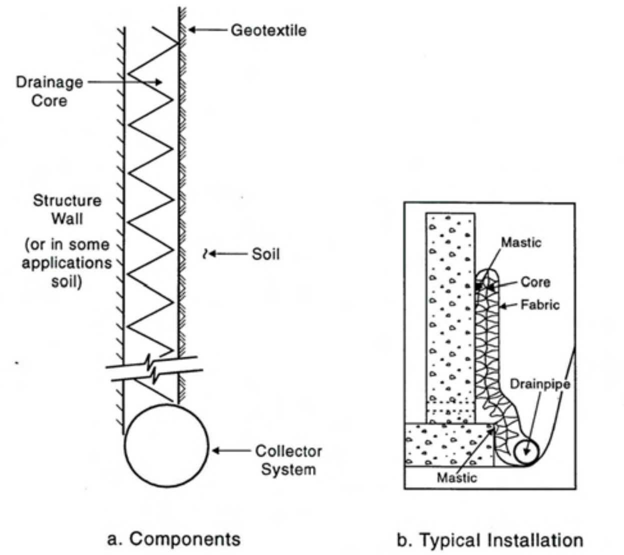


Figure 3.16. Geocomposite Drain (after Wahls, 1990).

Construction Methods

Construction methods can play a significant role in the development of the bump at the end of the bridge. The approach embankment can be constructed either before or after the bridge and the abutment. As described before, closed, spill-through, and integral abutments require the abutment first, but perched and MSE abutments are constructed after the embankment is finished. A typical cross section and construction sequence for a perched abutment is shown in Figure 3.17.

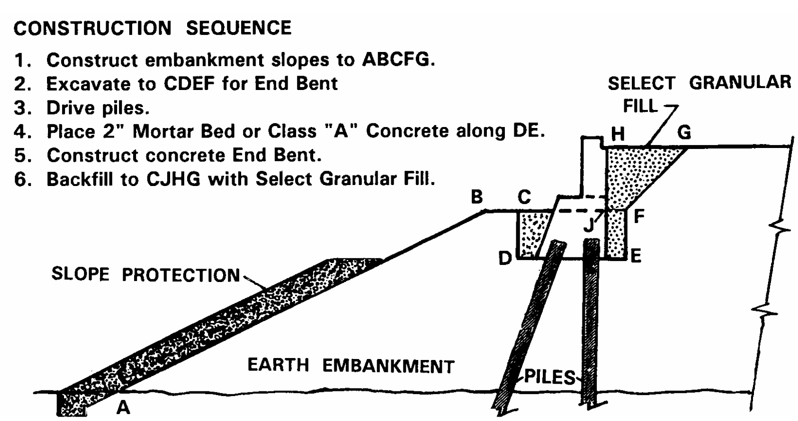


Figure 3.17. Example of Recommended Sequence for Embankment/Abutment Construction (after Hopkins, 1985).

Maintenance and Rehabilitation

Small movement of the abutments is inevitable but should not affect the performance of the bridge and approach system. Moulton et al. (1986) suggest a tolerable angular distortion (differential settlement between the ends of a span/span length) of 1/250 for continuous-span bridges and 1/200 for simply supported spans (Figure 3.18).

Preformed grout holes, physical jacking provisions, sleeper jacking provisions, pneumatic adjustable sleeper, and removable precast pavement panels have been considered to facilitate maintenance in the approach area for new construction. Mud-jacking, polyurethane jacking, overlay, and mechanical lifting of sleeper are currently available to repair existing bridge approaches. Figures 3.19 and 3.20 show the paved approach slab with asphalt roadway and the paved approach slab with concrete roadway, respectively.

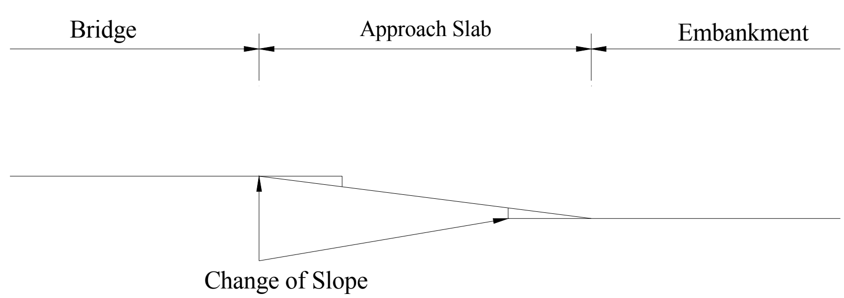


Figure 3.18. Definition of Approach Slab Slopes (after Wahls, 1990, and Burke, 1987).

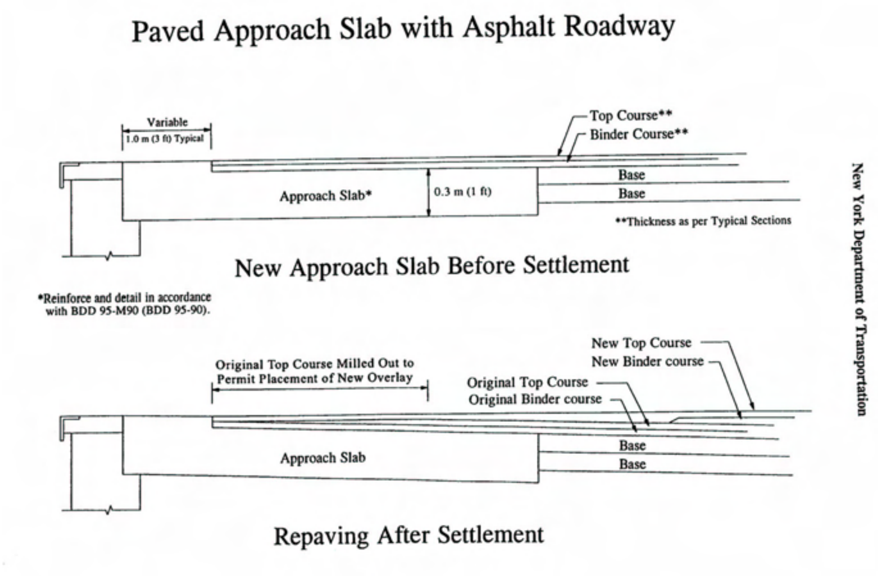


Figure 3.19. Paved Approach Slab with Asphalt Roadway (after Briaud et al., 1997).

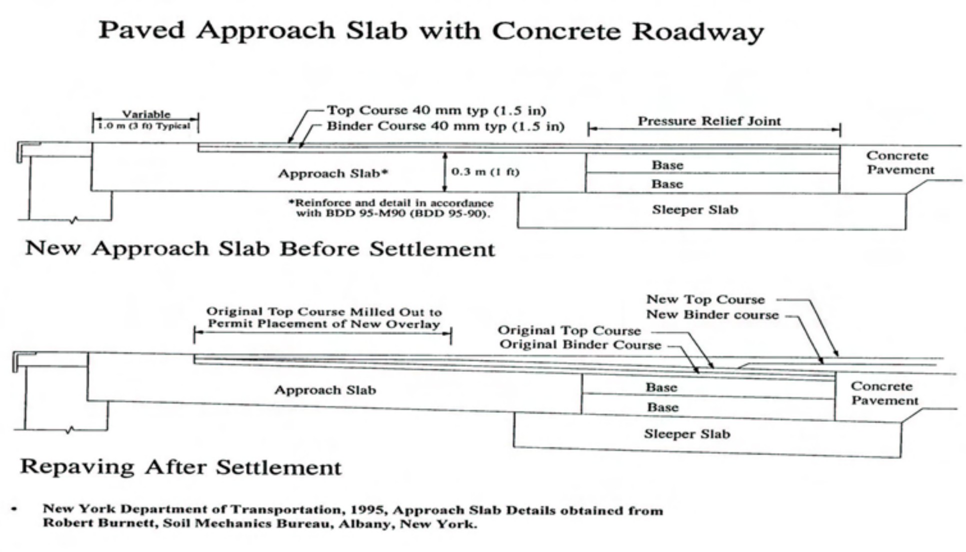


Figure 3.20. Paved Approach Slab with Concrete Roadway (after Briaud et al., 1997).

Current Practice in Houston, Texas

Abutment

Most bridges designed in Texas have “stub” or “perched” abutments as shown in Figures 3.21 and 3.6. Abutments must be compatible with the bridge approach roadway. They have backwalls to keep the embankment from covering up the beam ends and to support possible approach slabs. They usually have wingwalls to keep the sideslopes away from the structure and to transition between the guardrail and the bridge rail. The design of abutments with backwalls has been standardized through trial and error (TxDOT Bridge Design Manual, 2000) and is shown in Appendix D.

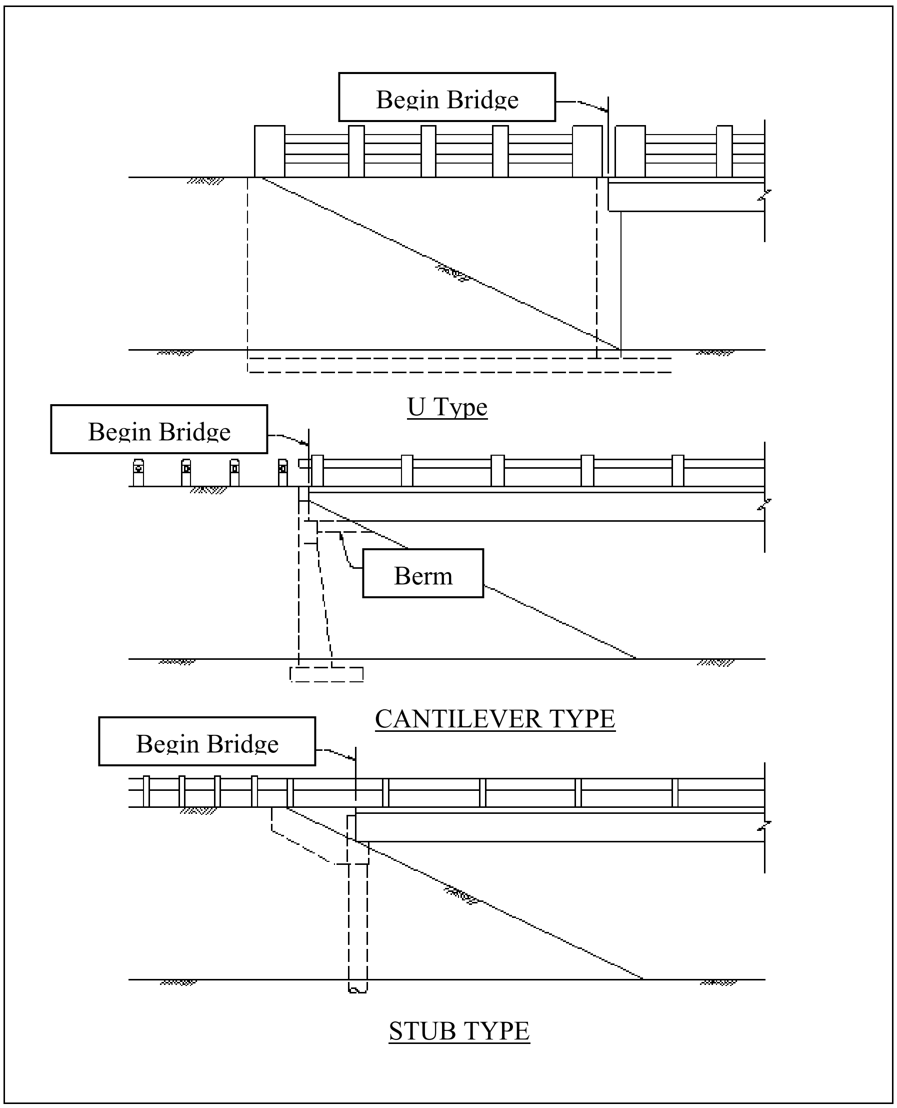


Figure 3.21. Stub Abutment (after TxDOT, 2001).

Wingwall

A wingwall is used to confine the abutment backfill material and roadway soil on each side of the side of the embankment, behind the abutment backwall. Wingwalls can be either cantilevered or founded. The limitation of the cantilevered wing wall is 12 ft. Wingwalls greater than 12 ft in length must be founded by drilled shaft(s) or pile(s). The TxDOT "Standard Details" for abutments including wingwall details are presented in Appendix D. Additional information can be found in the TxDOT Bridge Detailing Manual (http://manuals.dot.state.tx.us/dynaweb/ colbridg/des/@Generic__BookView).

Retaining Walls

Several types of walls may be used in conjunction with bridge abutments. In cut situations, the walls will often be cantilevered drilled shaft type walls, tied-back walls, or even spread footing type walls. The wall and bridge abutment will often become a single structure in these cases. Soil or rock nailed walls also may be used to support abutments in cut situations. In the most common situation, the walls will be mechanically stabilized earth (MSE) walls. Although the abutment cap can be placed directly on the MSE fill without deep foundations, this has not been a common practice in Texas; therefore, drilled shaft or piling foundations must be provided. The foundations are required to be installed prior to construction of the MSE wall, in order to avoid damage to the wall reinforcements during foundation installation (TxDOT Bridge Design Manual, 2000).

Approach Slabs

TxDOT uses 12-inch-thick approach slabs with lightly reinforced concrete that precede the abutment at the beginning of the bridge, and follow the abutment at the end of the bridge (Figures 3.22 and 3.23). The use of approach slabs is optional. The TxDOT Bridge Design Manual suggests that the approach slab should be supported by the abutment backwall and the approach backfill only. Therefore, an appropriate backfill material is essential. TxDOT is currently supporting the placement of a cement stabilized sand (CSS) "wedge" in the zone behind the abutment. CSS solves the problem of difficult compaction behind the abutment, and it is resistant to the moisture gain and loss of material that is common under approach slabs. The use of CSS has become standard practice in several districts and has shown good results (TxDOT Bridge Design Manual, 2000).

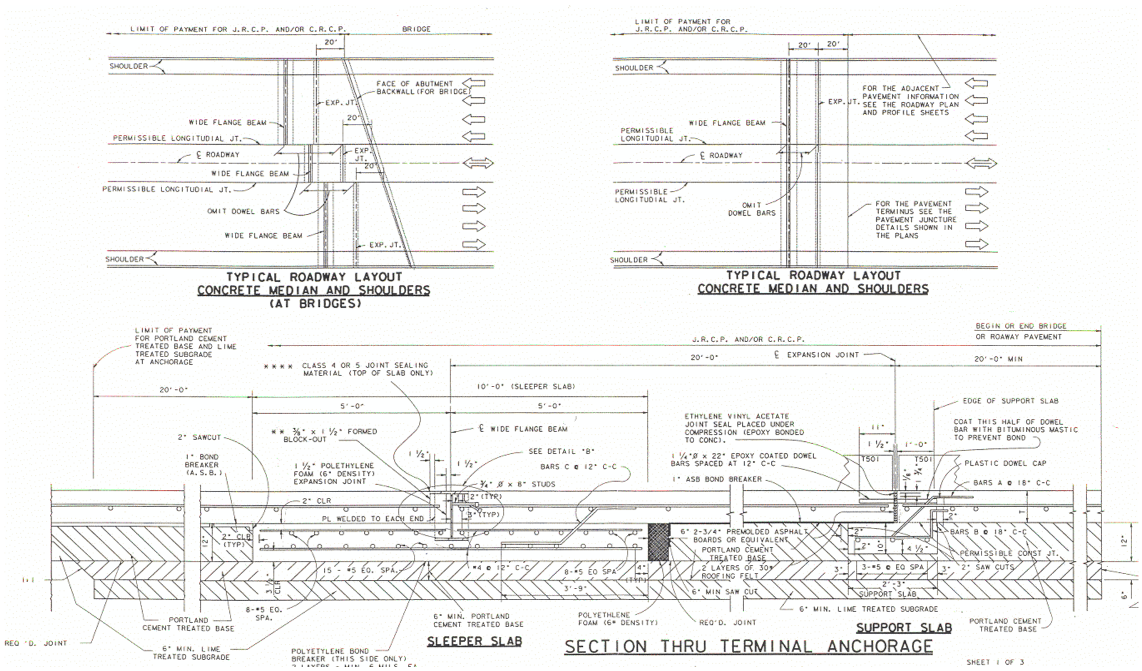


Figure 3.22. Approach Slab (after TxDOT, 1999).

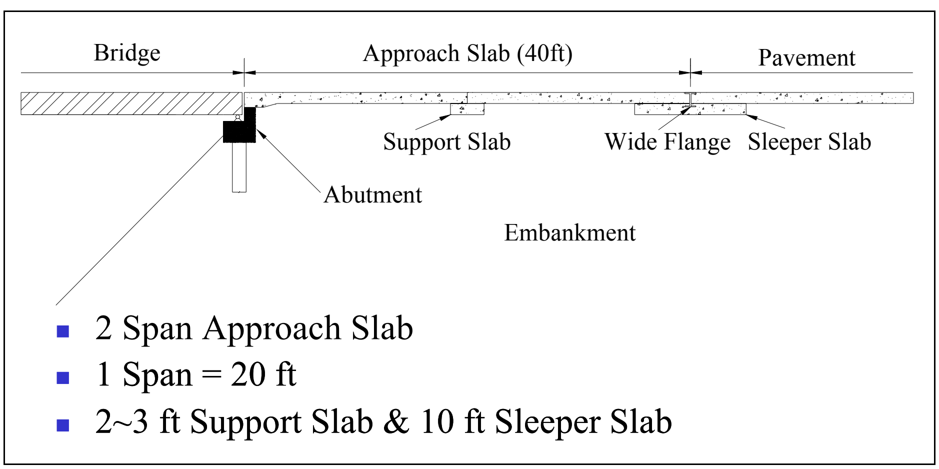


Figure 3.23. Sketch of Approach Slab.

Table 3.1 describes the design and construction of continuously reinforced concrete pavement (CRCP) construction joints and terminal in several states, including Texas.

Embankment

Suitable fill material is a soil with a liquid limit less than 45 percent, a plasticity index less than 15 percent, and a bar linear shrinkage more than 2 percent (Appendix E). The guide schedules for sampling and testing of embankment soils are also presented in Appendix F. It shows that the sampling locations are determined by the engineer and that the frequency of sampling is one test per 5,000 C.Y. for project tests and one test per 50,000 C.Y. for independent assurance tests.

Table 3.1. Design and Construction Feature of CRCP (after CRSI, 2000).

State	Texas	Illinois	Oklahoma	Oregon	South Dakota	Virginia
Design Procedure	Modified AASHTO	Modified AASHTO	AASHTO	AASHTO	AASHTO	AASHTO
Design Crack Width, inches	0.025	Not specified	0.04	0.04	0.04	Not specified
Slab Thickness, inches	8-15	10 (min. on interstate)	9-12	8-12	8-11	10-11
Outside Lane Width, inches	12	12	12	14	12 or 14	12 or 14
PCC Strength Measurement Method	28-day flexural 3rd point	14-day comp. & flexural strength	28-day Compressive	28-day Compressive	28-day Compressive	28-day Compressive
PCC Strength, psi	650 flexural	3,500 comp. 650 flexural	3,000 comp. (Class A PCC)	4,000 comp.	4,000 comp.	3,000 comp.
Primary Aggregate Type	Limestone, and siliceous river gravel	Gravel, crushed gravel, stone, concrete, slag or sandstone	Crushed limestone	Crushed basalt	Quartzite, limestone, granite	Various non-polished
Max. Aggregate Size, inches	0.75-1.5	1.5	1.5	1.5	1.0	AASHTO 357 (100% passing 2.0-in. sieve)
PCC Curing Method	2 coats of curing compound	Wet cure or type III cur. Comp.	White resin based wax curing comp.	Curing compound	White pigmented curing compound	Curing compound
Placement Season	All year	Not specified	All year (except extreme cold)	All year	Spring, summer, fall	Spring, summer, fall
Placement Time of Day	Day or night	Not specified	Day	Day or night	Day	Day
Base Type(1)	CTB with HMA breaker, ATB	BAM	ATB, OGPB, Econocrete	ATB or Granular	Granular, CTB with HMA breaker, ATB	CTB
Permeable Base	No	No	Sometimes	Sometimes	No	Yes
Base Thickness, inches	CTB: 6 ATB: 4	4	4	ATB: 4 Granular: 6	Granular: 6	6-8
Improved Subgrade	6-8 in. lime stabilization	Line modification	Stabilization	Remove prob. areas & fill w/granular mat'l	None	Occasionally soil cement stabilization
Outside Shoulder Type	Same as travel lane	PCC	Plain PCC (doweled in urban areas)	AC	AC or PCC	AC or PCC
Amount of Longitude Steel, %	8-in. slab: 0.4-0.5 15-in. slab: 0.71-0.78	0.7	0.71-0.73	0.6-0.7	0.7	0.7

Table 3.2. Design and Construction Feature of CRCP (cont.).

State	Texas	Illinois	Oklahoma	Oregon	South Dakota	Virginia
Steel Grade, ksi	60	Long: 60 Transv.: 40 or 60	60	60	60	60
Steel Placement Method	Chairs	Chairs	Chairs or tube-fed	Chairs	Chairs	Chairs
Epoxy Coated Steel	No	In Chicago area only	Urban: yes Rural: no	No	No	No
Depth of Steel (from slab surface), inches	Mid-slab (2 layers if > 13" thick)	3.5	Mid-slab	4.0	3.0-4.0	Mid-slab (±0.5 inches)
Amount of Transverse Steel	#5 or #6 bars at 30-36 in. spacing	#4 bars at 48-in. spacing (0.04% max)	#5 bars at 44-in. spacing	#4 bars at 36-in. spacing	0.15%	#5 bars at 48-in. spacing
Construction Joint Design	If slab ≤ 9 inches then(2)	Additional #6 bars (2)	(2)	No extra steel	Additional #6 bars (2)	(2)
Terminal Design	Occasionally use anchor lugs, but moving toward wide-flange beam	Wide-flange beam	Sleeper slab	Wide-flange beam	Manufactured beam embedded in a sleeper slab	Anchor slab

(1) BAM = Bituminous-Aggregate Mix; ATB = Asphalt-Treated Base; OGDB = Open-Graded Drainable Base; CTB = Cement-Treated Base; HMA = Hot Mix Asphalt

(2) Additional 72-inch-long bars placed adjacent to every other longitude bar (same diameter as longitudinal steel), unless noted.

CHAPTER 4: NUMERICAL ANALYSIS

The purpose of the numerical analyses was to evaluate the behavior of the current approach slab and of a possibly more effective approach slab. The researchers used ABAQUS to simulate the behavior of the transition zone including the bridge abutment, the approach slab, and the embankment. The first section of this chapter covers the assumptions and the model used. The second section describes the results. A summary of the results is presented in the third section.

Assumptions and Model

One of the most important steps in numerical simulations is to determine where the boundaries should be placed. Normally the bottom of the mesh is the depth of a notably harder soil. In this project, it was assumed that the hard boundary is located 7 m below the bottom of the fill. This value came from the CPTs done during the first year work at two Houston sites. Indeed the tip resistance of the CPT at that depth increased significantly. Briaud and Lim (1997) recommended boundary distances for the simulation of the removal of the embankment soil-wedge in front of the abutment on piles and the nailing of the exposed vertical force. Figure 4.1 shows their recommendations and results. The horizontal distance from the wall face to the mesh boundary at the end of the embankment is B_e, and W_e is the horizontal distance from the wall face to the other end of the mesh. D is the distance from the bottom of the excavation to the hard layer, and H_e is the height of the soil-wedge to be removed. For a given D and H_e, it was found that when W_e increased beyond 3D and B_e increased beyond $3(H_e + D)$, the horizontal deflection at the top of the wall due to the removal of the soil wedge only increased by a few percent. Therefore, since in this research project $D = H_e = 7$ m, a W_e of 21 m and B_e of 42 m were used for all simulations.

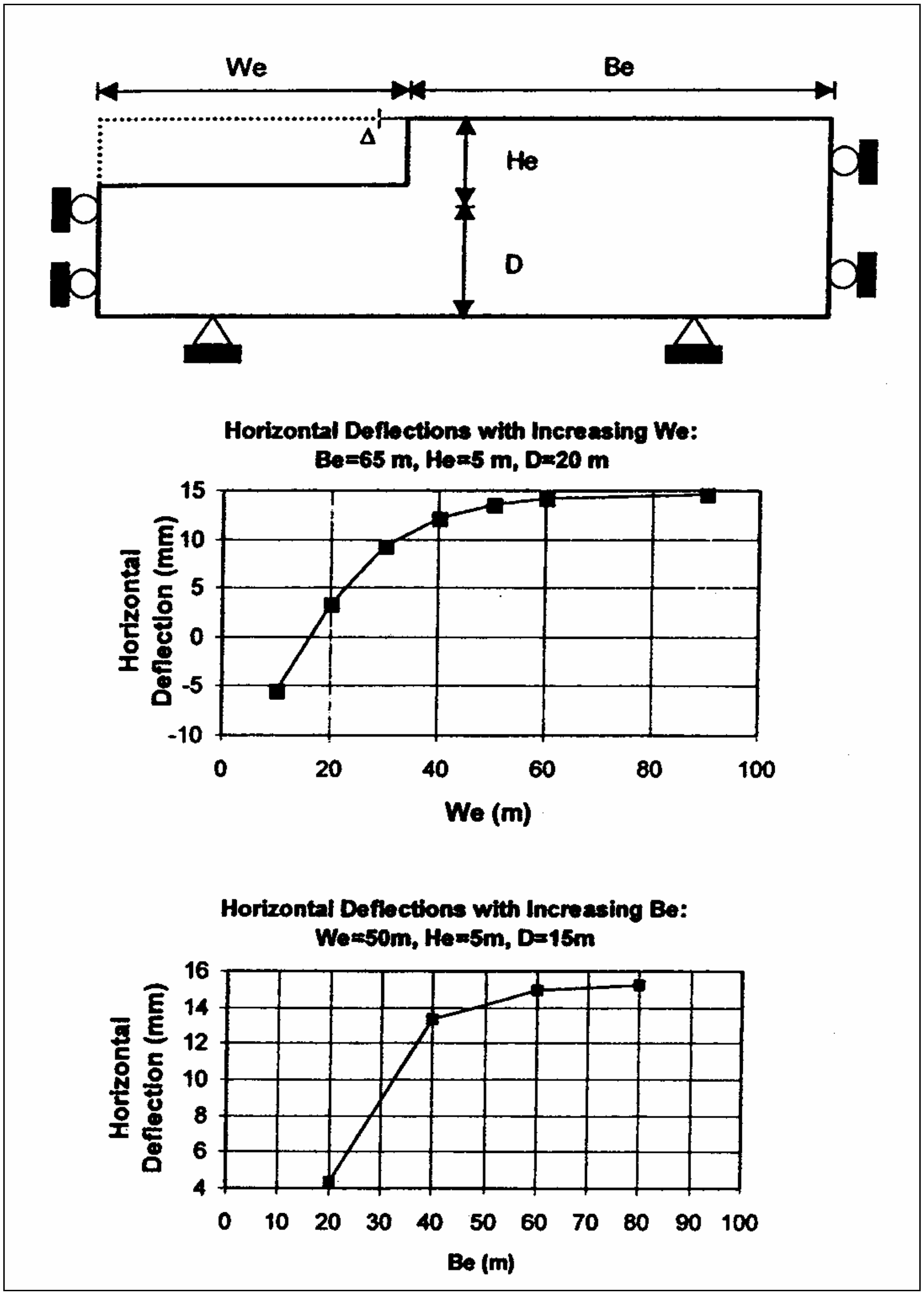


Figure 4.1. Influence of Mesh Size on Horizontal Deflection (after Briaud and Lim).

Figure 4.2 shows a finite element model to simulate the bump at the end of the bridge. A schematic of the approach slab is shown in Figure 4.3. This model was simplified by employing elastic materials with a plain strain condition. The bottom of the model was a fixed boundary. The left and right sides of the model were on vertical rollers and were restrained horizontally. The top of the abutment was also placed on rollers because the bridge prevents the movement of pavement. All the analyses were done with static loads. Four loading cases were applied to the model. Three loading cases (case 1, case 2, and case 3) consisted of a 100 kN point load placed at the center of the support slab, at the center of the sleeper slab, and 27 m away from the abutment wall, respectively, and one loading case (Case 4) consisted of a uniform load placed on top of the pavement. Figure 4.4 shows the material zones and loading cases. Several permutations of modulus values were used in zone 3 (Figure 4.4) to simulate different soil conditions. The modulus values for the various zones of Figure 4.4 are shown in Table 4.1 along with Poisson's ratio.

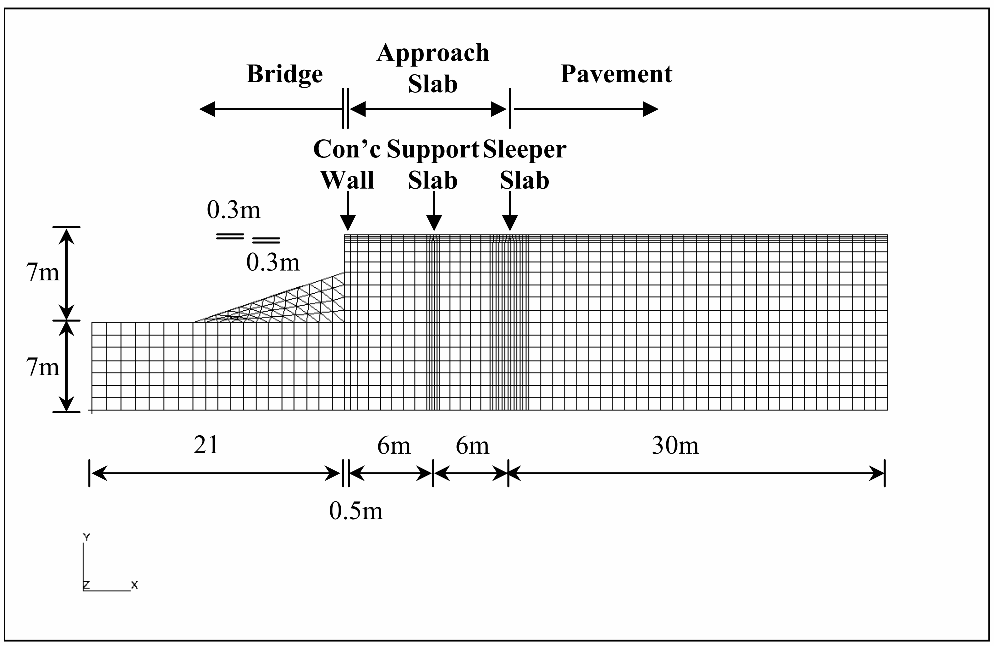


Figure 4.2. Finite Element Model.

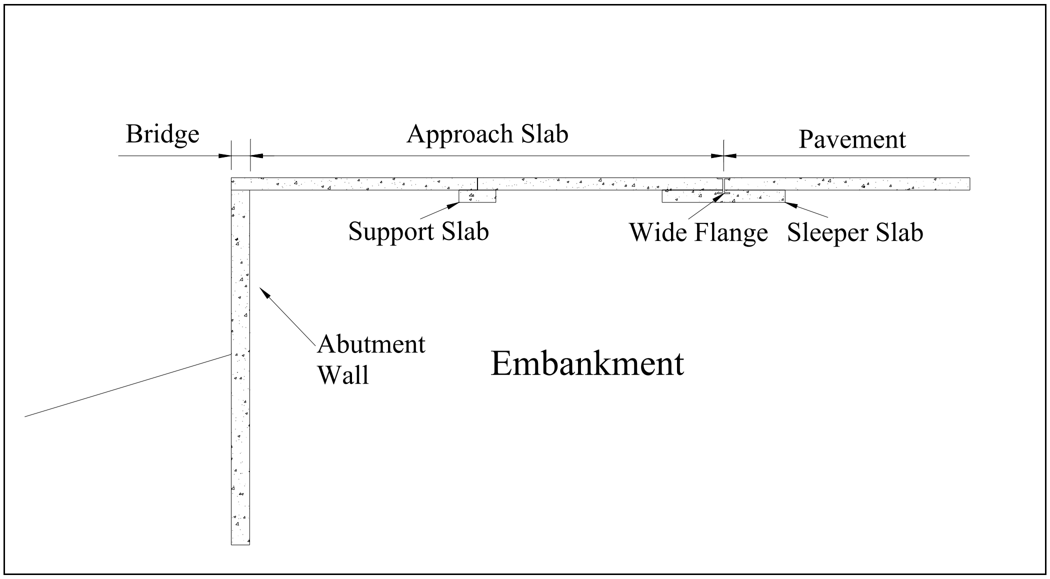


Figure 4.3. A Schematic of the Approach Slab.

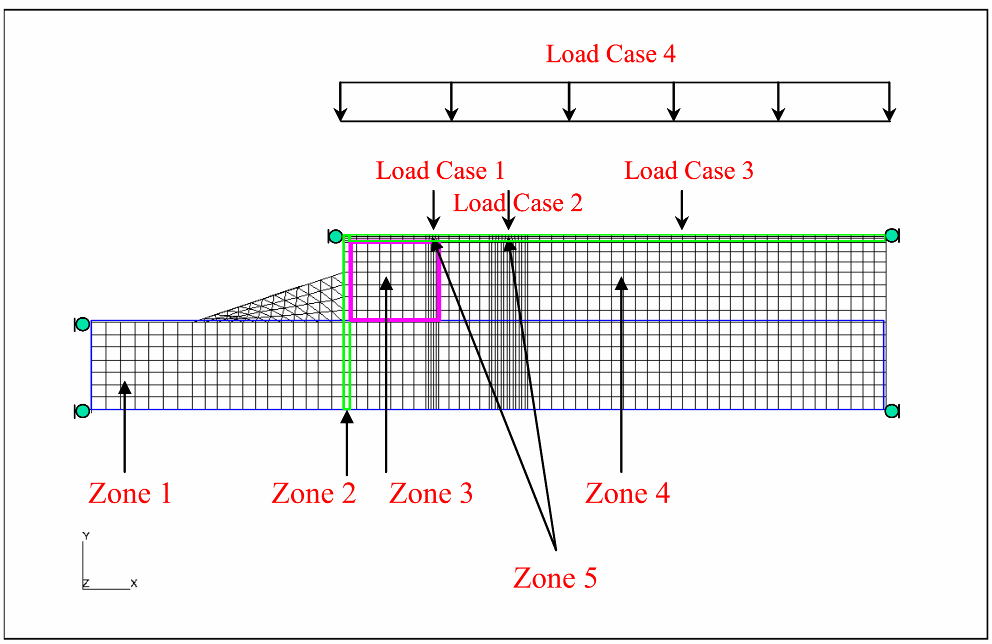


Figure 4.4. Zones and Load Cases of the Finite Element Model.

Table 4.1. Material Properties.

Material	Young's Modulus	Poisson's Ratio	Zone
Fill Soil	10×10^3 kPa	0.35	4
Natural Soil	20×10^3 kPa	0.35	1
Weak Soil	2.5×10^3 kPa	0.35	3
Soft Soil	5×10^3 kPa	0.35	3
Stiff Soil	10×10^3 kPa	0.35	3
Concrete Pavement	2×10^7 kPa	0.30	2
Approach Slab	2×10^7 kPa	0.30	2
Expansion Joint	2×10^3 kPa	0.35	5

For verification purposes, a simple rectangular model was subjected to a pressure of 100 kPa as shown in Figure 4.5. The numerical result was compared with the theoretical solution. A displacement of 0.043 m was calculated using equations (4.1) to (4.8). The numerical result also gave 0.043 m as shown on Figure 4.5.

$$\varepsilon_z = \frac{1}{E}\{\sigma_z - \nu(\sigma_x + \sigma_y)\} \tag{4.1}$$

$$\varepsilon_x = \frac{1}{E}\{\sigma_x - \nu(\sigma_y + \sigma_z)\} \tag{4.2}$$

$$\varepsilon_y = \frac{1}{E}\{\sigma_y - \nu(\sigma_x + \sigma_z)\} \tag{4.3}$$

$$\varepsilon_x = 0, \quad \sigma_x = \nu(\sigma_y + \sigma_z) \tag{4.4}$$

$$\varepsilon_y = 0, \quad \sigma_y = \nu(\sigma_x + \sigma_z) \tag{4.5}$$

$$\sigma_x + \sigma_y = 2\nu\sigma_z + \nu(\sigma_x + \sigma_y) = \frac{2\nu}{1-\nu}\sigma_z \tag{4.6}$$

$$\varepsilon_z = \frac{\Delta H}{H} = \frac{\sigma_z}{E}\{1 - \frac{2\nu^2}{1-\nu}\} \tag{4.7}$$

$$\Delta H = H\frac{\sigma_z}{E}\{1 - \frac{2\nu^2}{1-\nu}\} = 14\frac{100}{20000}\{1 - \frac{2\times0.35^2}{1-0.35}\} = 0.043(m) \tag{4.8}$$

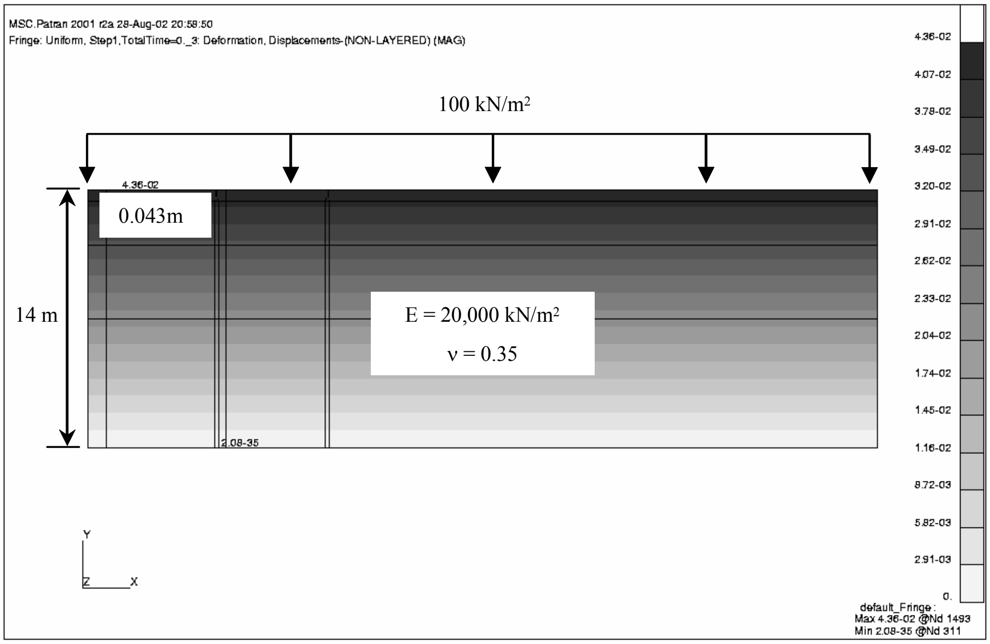


Figure 4.5. Numerical Verification.

Numerical Simulation Results

Using the finite element model described above, the research team simulated several cases. The thickness of the wall, the stiffness of the soil, the height of the embankment, and the length of the slab were changed to study their influence on the bump at the end of the bridge. A total of 36 analyses were done and the results are shown in Appendix G.

Thickness of Wall and Stiffness of Soil

Three different thicknesses of abutment wall (Figure 4.3) (no wall, 0.5 m wall, and 1.0 m wall) were considered to study their effect on the settlement of the approach slab. There is a differential settlement between the bridge abutment and the embankment soil because the settlement of the bridge abutment, which is usually supported on piles, is smaller than the settlement of the embankment. The effect of the wall thickness on this differential settlement was studied in this section. Figures 4.6 to 4.8 show the deformed meshes for a soft soil in zone 3 (Figure 4.4 and Table 4.1) (Young's modulus of 5,000 kPa and load case 4). The settlement profiles for the soft soil case are shown in Figure 4.9. As described in the first year report, the stiffness of the soil near the abutment was quite different from that away from the abutment. In this section, three different soils stiffnesses, 2,500 kPa, 5,000 kPa, and 10,000 kPa, were also considered in zone 3 (Figure 4.4 and Table 4.1) to study the effect of soil stiffness on the settlement. Typical deformed meshes for load case 4 are shown in Figures 4.10 to 4.12. The settlement profiles for a 0.5 m wall thickness are shown in Figure 4.13.

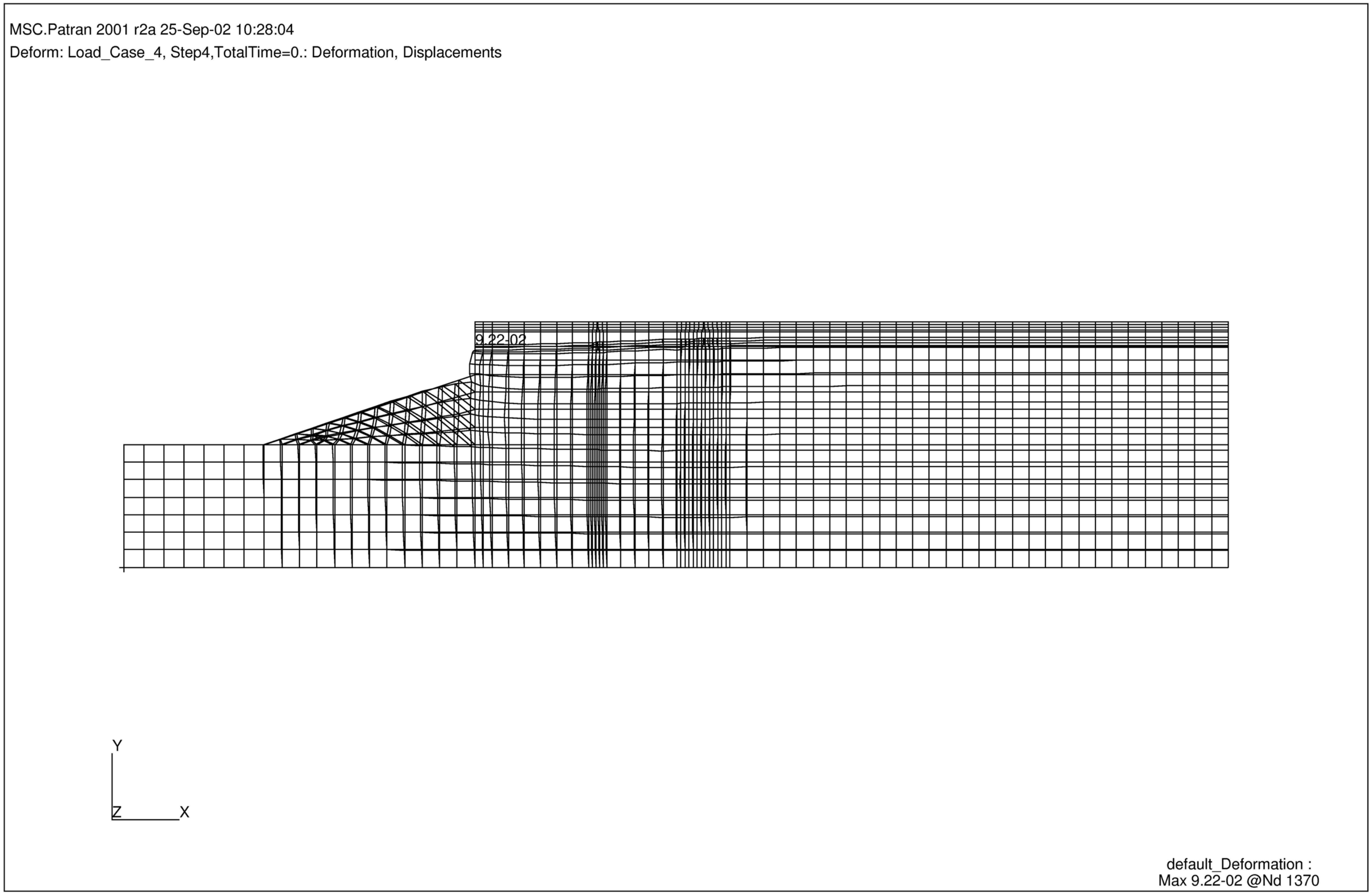


Figure 4.6. No Wall, Load Case 4, and E = 5,000 kPa in Zone 3.

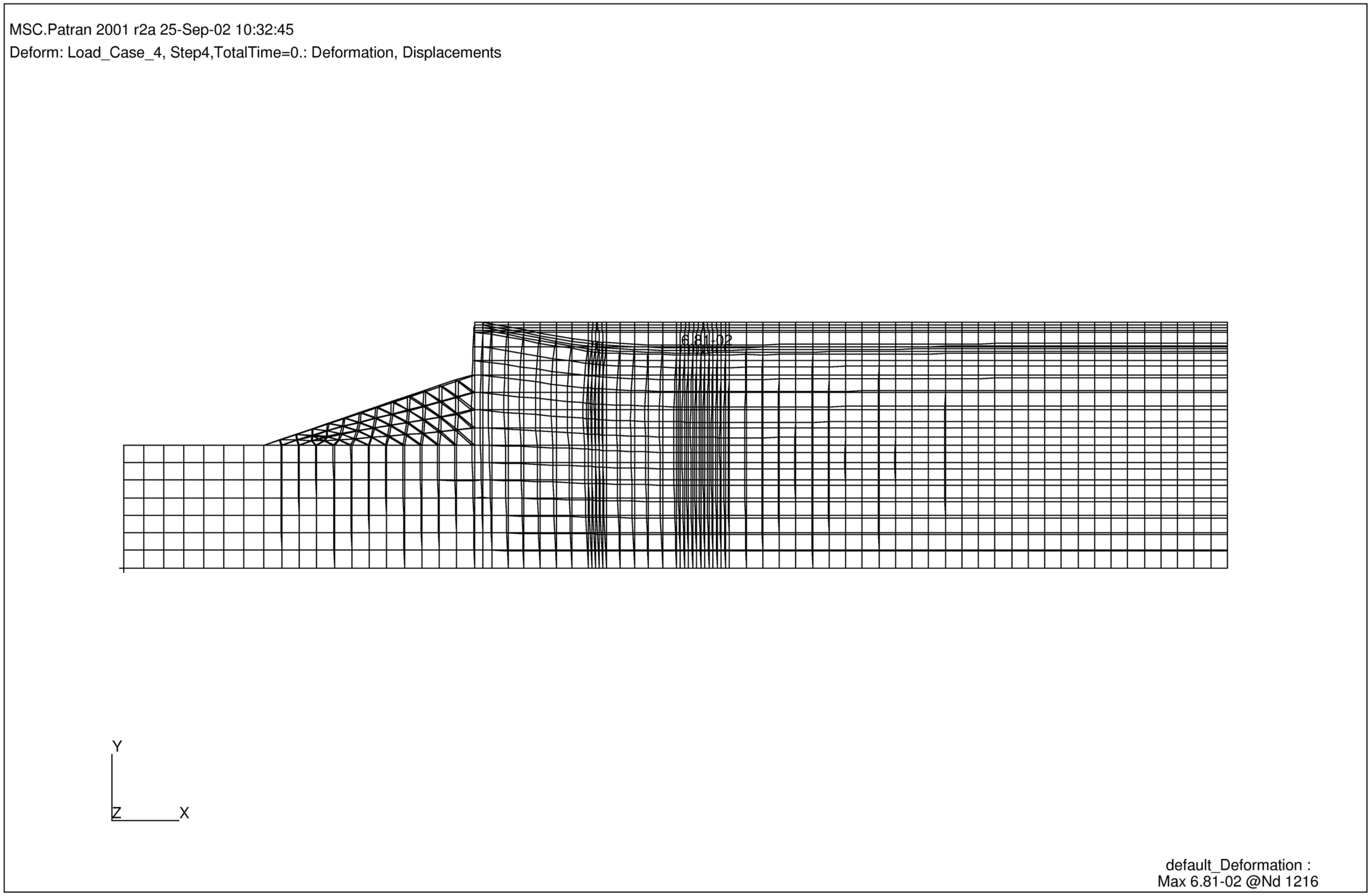


Figure 4.7. 0.5 m Wall, Load Case 4, and E = 5,000 kPa in Zone 3.

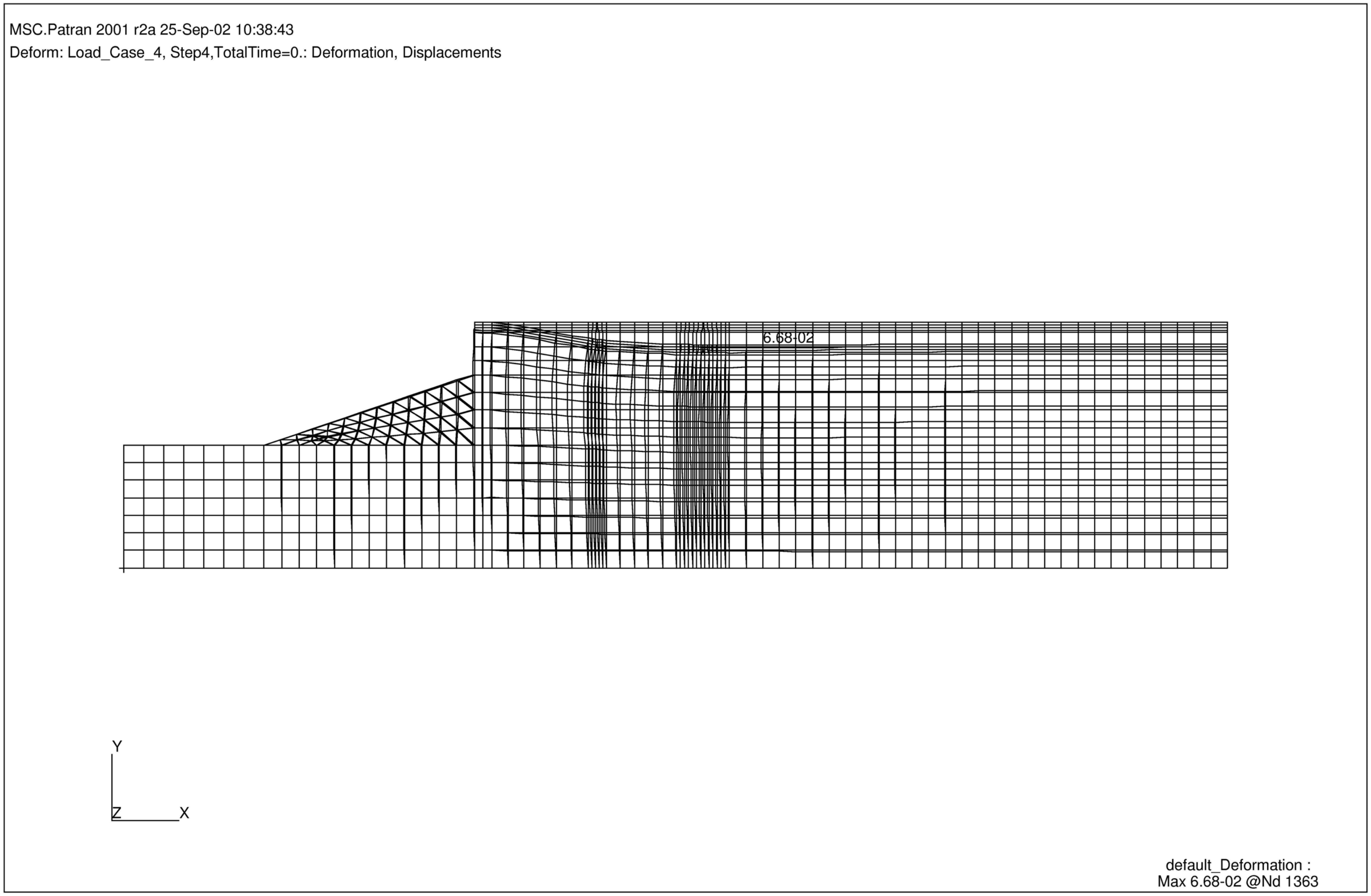


Figure 4.8. 1.0 m Wall, Load Case 4, and E = 5,000 kPa in Zone 3.

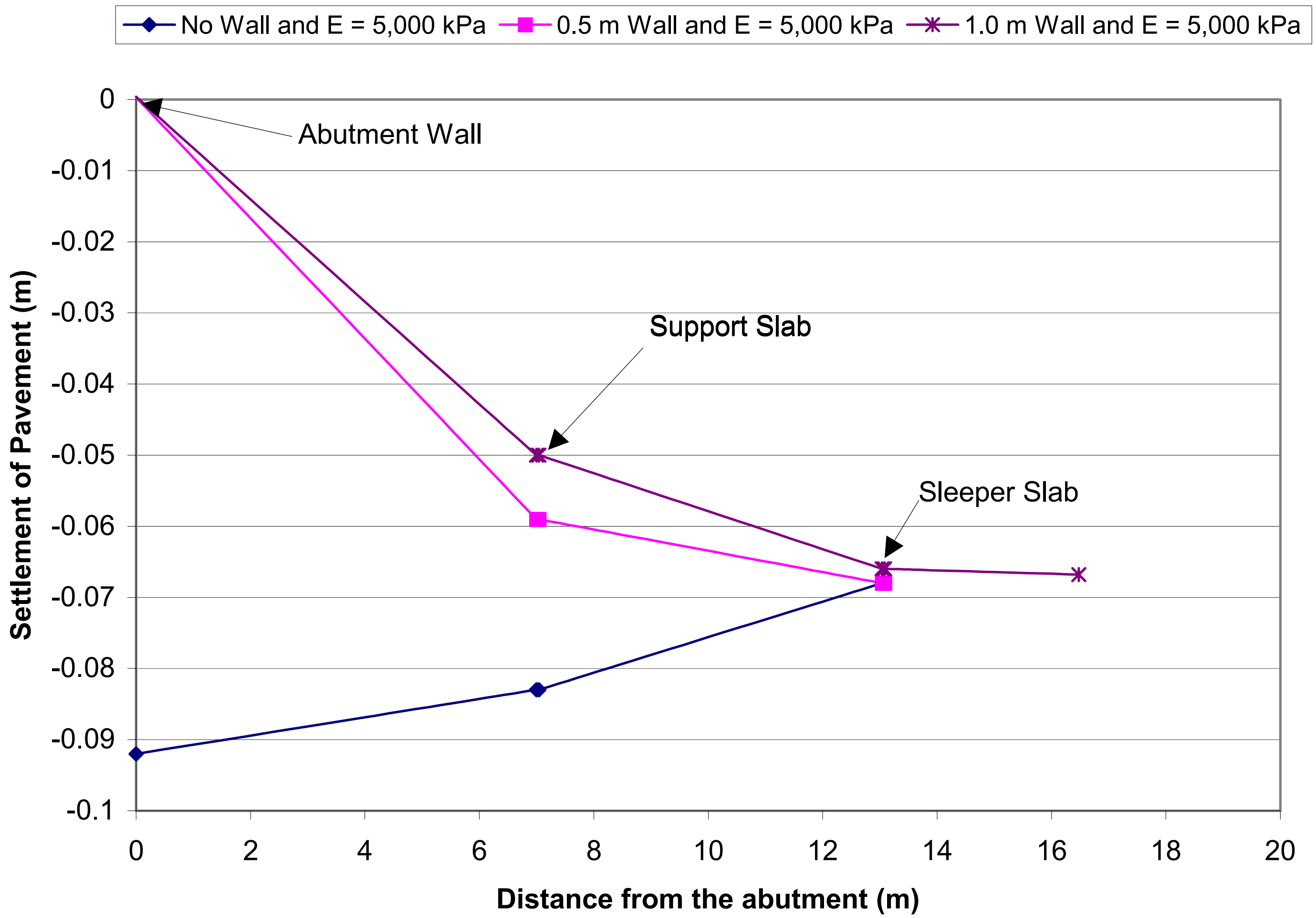


Figure 4.9. Settlement Profile for Three Different Walls (Load Case 4).

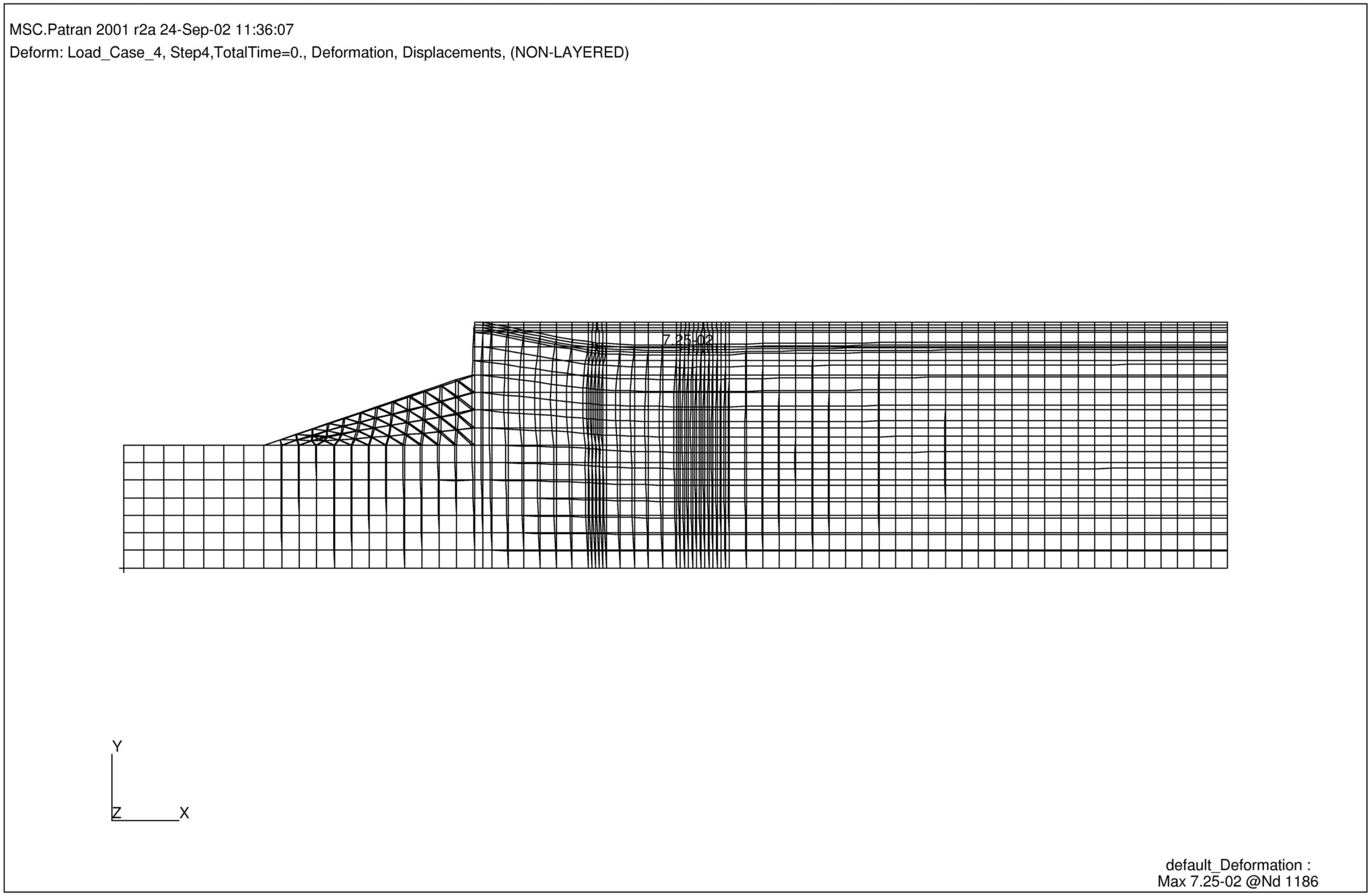


Figure 4.10. 0.5 m Wall, Load Case 4, and E = 2,500 kPa in Zone 3.

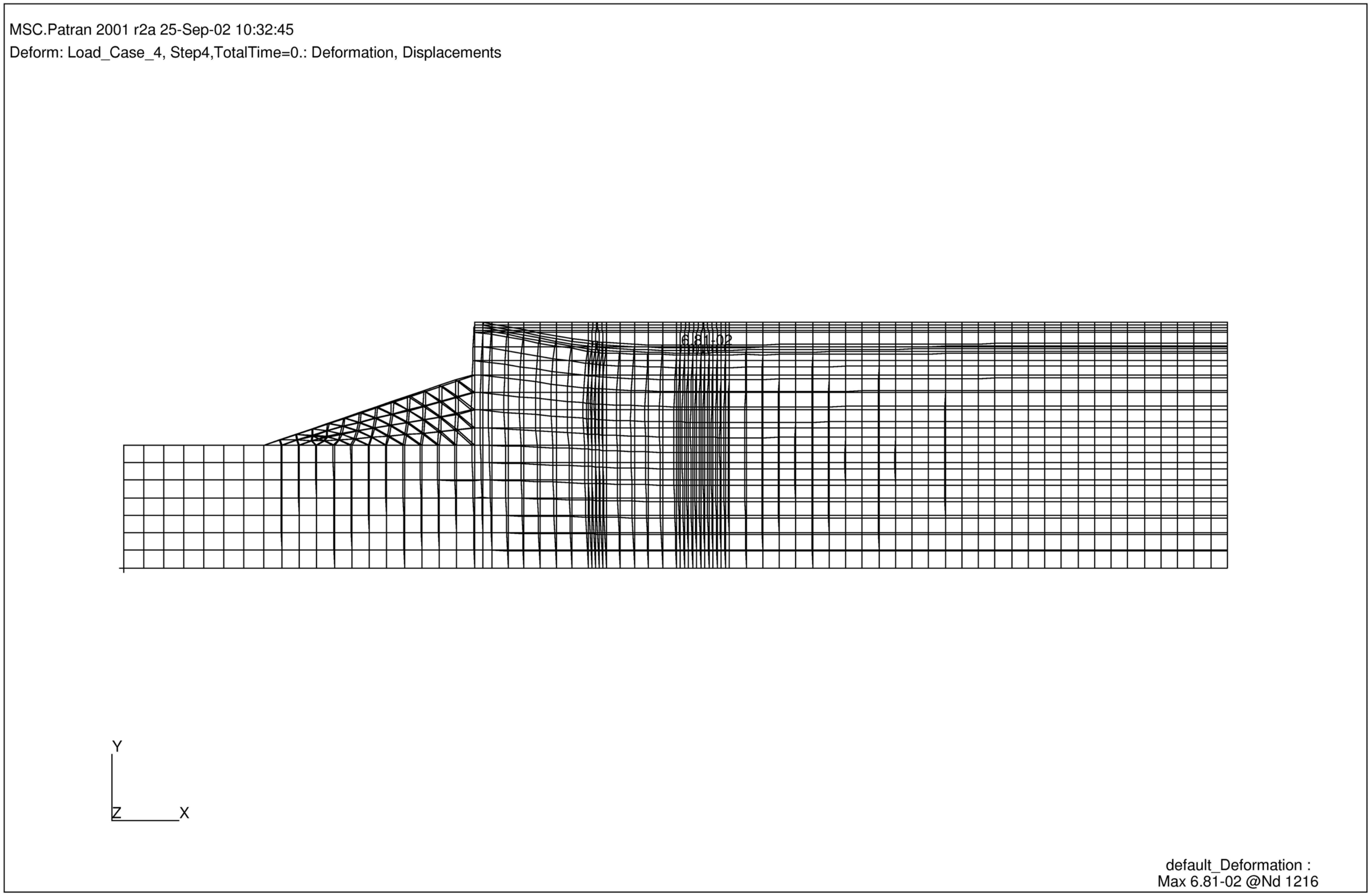


Figure 4.11. 0.5 m Wall, Load Case 4, and E = 5,000 kPa in Zone 3.

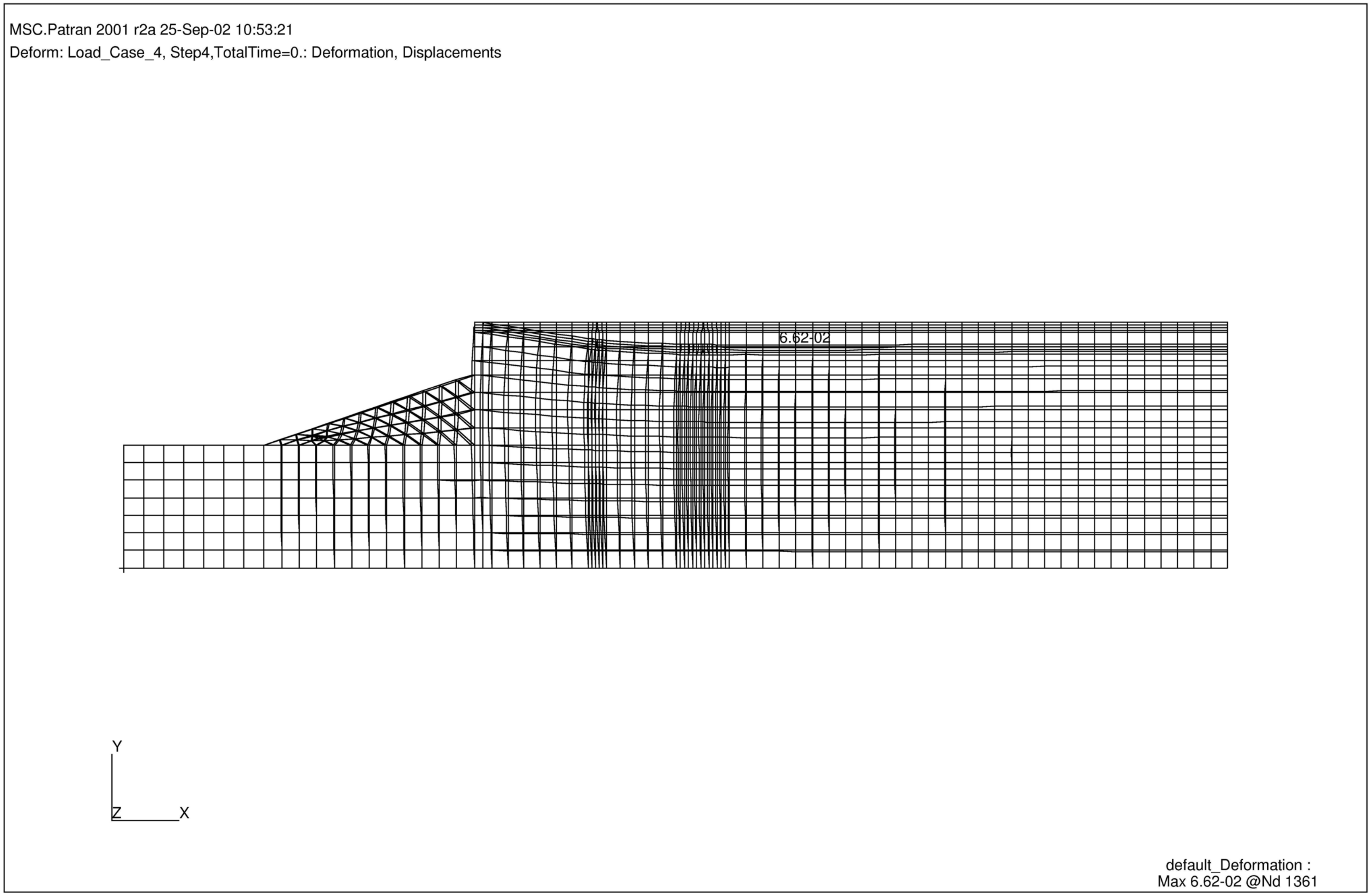


Figure 4.12. 0.5 m Wall, Load Case 4, and E = 10,000 kPa in Zone 3.

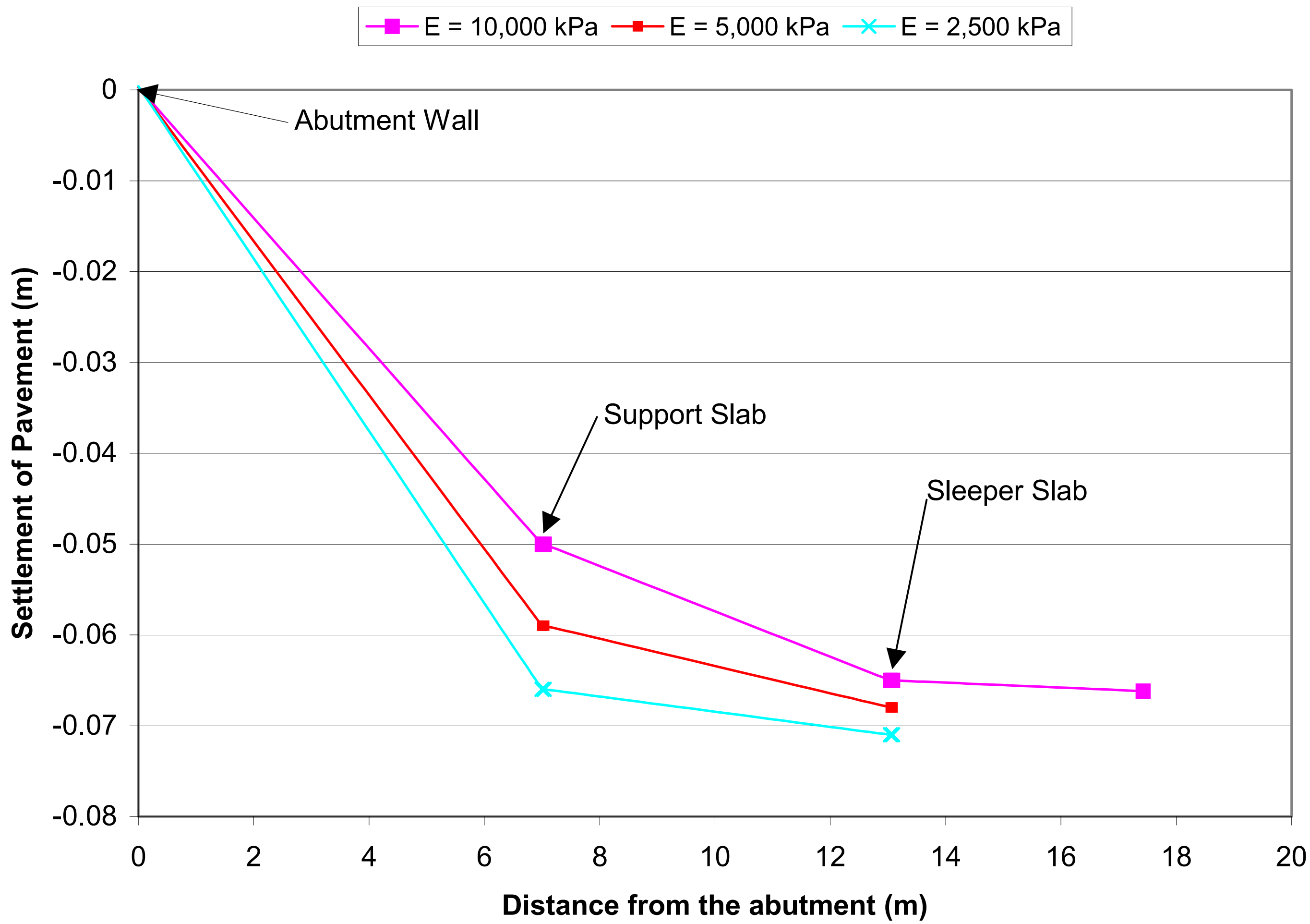


Figure 4.13. Settlement Profile for Three Different Soil Moduli in Zone 3 (Table 4.1).

Δ_1 and Δ_2 are the gradients of the slope between the abutment and the support slab and the support slab and sleeper slab, respectively, as shown in Figure 4.14. The numerical results for the three different walls and three different soils conditions are summarized in Table 4.2. As can be seen in Table 4.2, the biggest bumps are developed when load case 4 is applied to the pavement, and the smallest bumps are developed when there is no wall. The results also show that the bumps decreased when the stiffness of the soil in zone 3 increased.

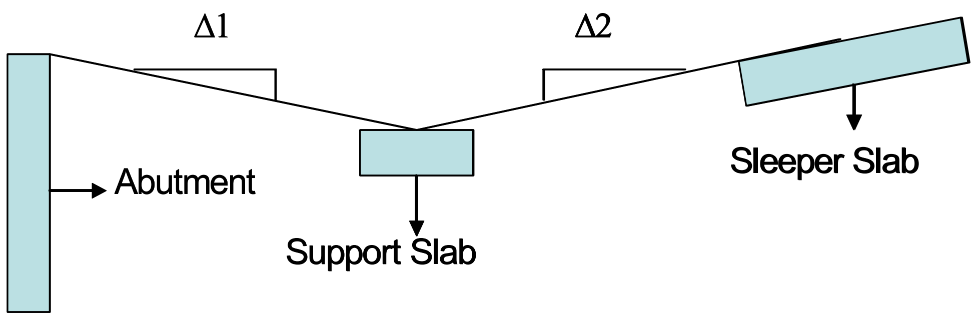


Figure 4.14. Gradient of Slope.

Table 4.2. Summary of Results (See also Table 4.1 and Figure 4.4).

(a) No Wall

Loading Case	Weak Soil in Zone 3		Soft Soil in Zone 3		Stiff Soil in Zone 3	
	Δ_1	Δ_2	Δ_1	Δ_2	Δ_1	Δ_2
Case 1	-0.04/100	0.13/100	-0.06/100	0.10/100	-0.06/100	0.07/100
Case 2	-0.04/100	-0.06/100	-0.03/100	-0.07/100	-0.04/100	-0.07/100
Case 3	-0.12/100	-0.03/100	0.01/100	-0.04/100	-0.01/100	-0.04/100
Case 4	0.57/100	0.63/100	0.13/100	0.23/100	-0.12/100	-0.02/100

(b) 0.5 m Wall

Loading Case	Weak Soil in Zone 3		Soft Soil in Zone 3		Stiff Soil in Zone 3	
	Δ_1	Δ_2	Δ_1	Δ_2	Δ_1	Δ_2
Case 1	-0.12/100	0.08/100	-0.10/100	0.07/100	-0.09/100	0.06/100
Case 2	-0.04/100	-0.06/100	-0.04/100	-0.07/100	-0.03/100	-0.07/100
Case 3	-0.00/100	-0.04/100	0.00/100	-0.04/100	0.00/100	-0.04/100
Case 4	-0.94/100	-0.08/100	-0.84/100	-0.15/100	-0.72/100	-0.23/100

Table 4.2. Summary of Results (cont.).

(c) 1 m Wall

Loading Case	Weak Soil in Zone 3		Soft Soil in Zone 3		Stiff Soil in Zone 3	
	Δ_1	Δ_2	Δ_1	Δ_2	Δ_1	Δ_2
Case 1	-0.10/100	0.07/100	-0.09/100	0.06/100	-0.08/100	0.05/100
Case 2	-0.79/100	-0.22/100	-0.03/100	-0.07/100	-0.03/100	-0.07/100
Case 3	-0.04/100	-0.07/100	0.03/100	-0.07/100	-0.03/100	-0.07/100
Case 4	-0.79/100	-0.22/100	-0.71/100	-0.25/100	-0.63/100	-0.30/100

Height of Embankment

The height of the embankment influences the bump at the end of the bridge. In this section, two different heights of embankment with 0.5 m wall thickness are chosen to evaluate the effect: a high approach embankment of 6.4 m and a low approach embankment of 3 m. Table 4.3 shows the settlement results. The deformed meshes are shown in Figure 4.15 and 4.16.

Table 4.3. Settlements for Different Embankment Height.

Embankment Type	Maximum Settlement (m) of Pavement Profile for 0.5 m Wall, Loading Case 4, and Soft Soil in Zone 3
Low Embankment (H_1=3 m)	$S_1 = 5.22\times10^{-2}$
High Embankment (H_2=6.4 m)	$S_2 = 6.82\times10^{-2}$

The model height includes the height of the embankment and the height of the natural soil (7 m in the model). Table 4.3 shows that the ratio of model heights ($(H_1+7)/(H_2+7)=1.34$) is close to the ratio of settlement ($S_1/S_2= 1.31$) as can be expected.

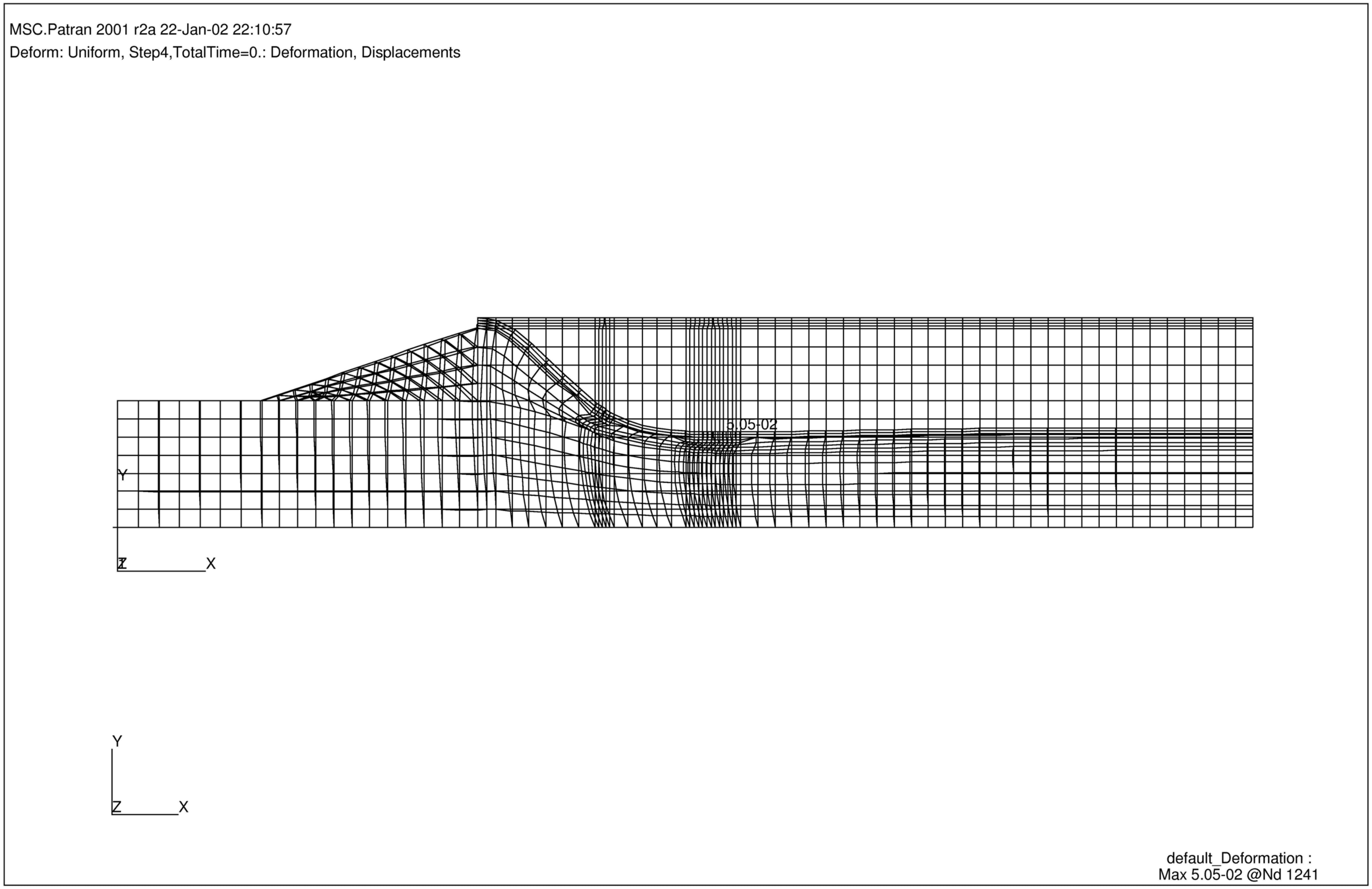


Figure 4.15. Settlement Profile of Low Embankment (H_1=3 m) Embankment.

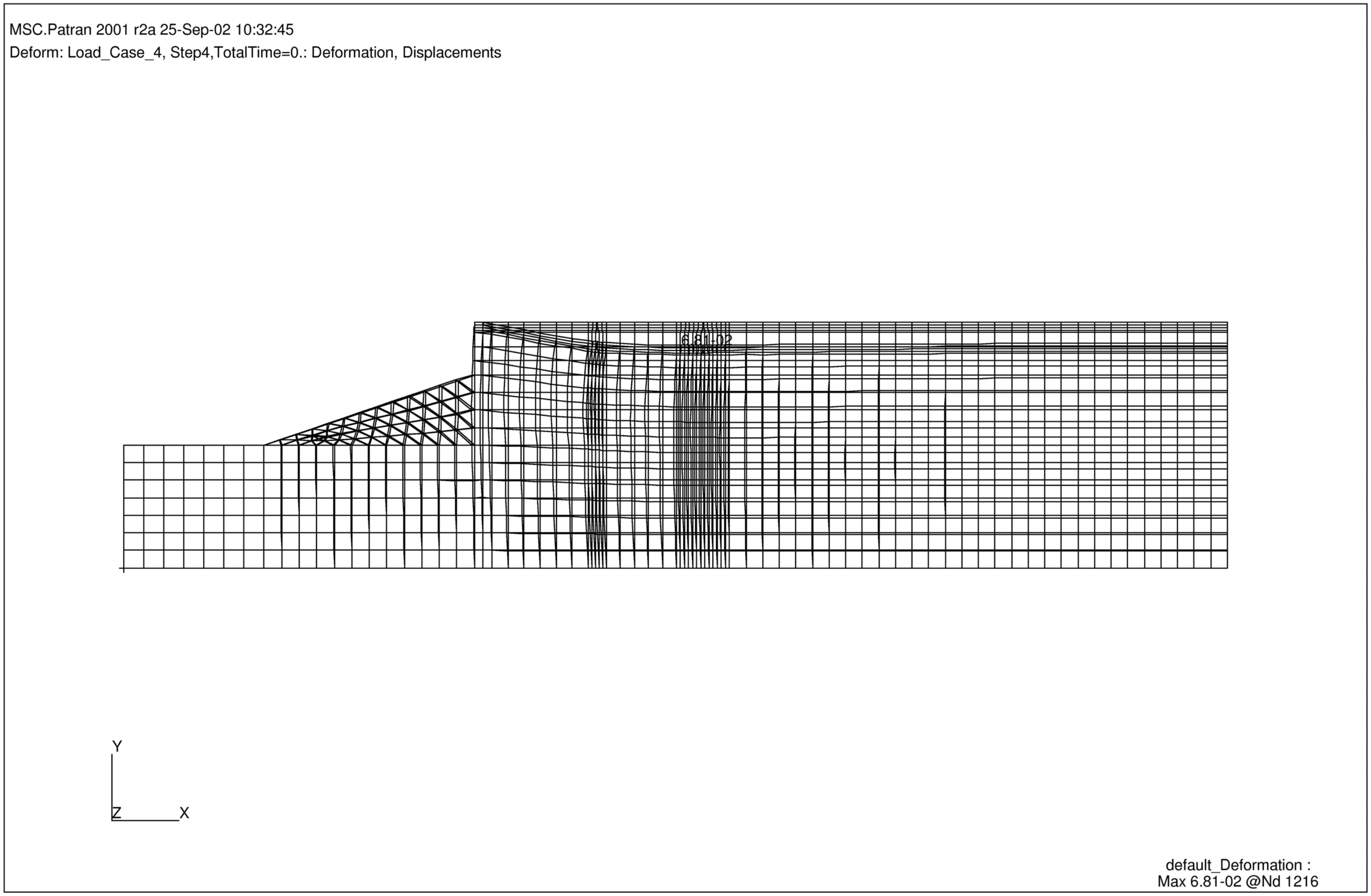


Figure 4.16. Settlement Profile of High Embankment (H_2=6.4 m) Embankment.

Length of Slab

The two-span approach slab is supported by two slabs: the support slab and the sleeper slab (Figure 4.3). The lengths of the support slab and of the sleeper slab underneath the pavement can influence the bump size. The researchers used various support and sleeper slab lengths to study their influence on the settlement of the support slab and the sleeper slab. The loading case was case 1 for the support slab and case 2 for the sleeper slab (Figure 4.4) and the soil in zone 3 was the soft soil (Table 4.1). Table 4.4 and Figure 4.17 show the results of the simulations. The settlement decreases as the slab length increases because the pressure on the soil decreases. Figure 4.17 shows that an optimum length for the support slab and for the sleeper slab is about 5 ft.

Table 4.4. Settlements as a Function of the Length of Slab.

Length of Support Slab (m)	Max. Settlement (m) of the Pavement	Length of Sleeper Slab (m)	Max. Settlement (m) of the Pavement
0.00	0.0125	0.00	0.0113
0.20	0.0105	0.23	0.0098
0.60	0.0081	0.69	0.0083
1.00	0.0068	1.15	0.0077
3.12	0.0056	1.62	0.0074
-	-	2.08	0.0072
-	-	2.54	0.0069
-	-	3.00	0.0067

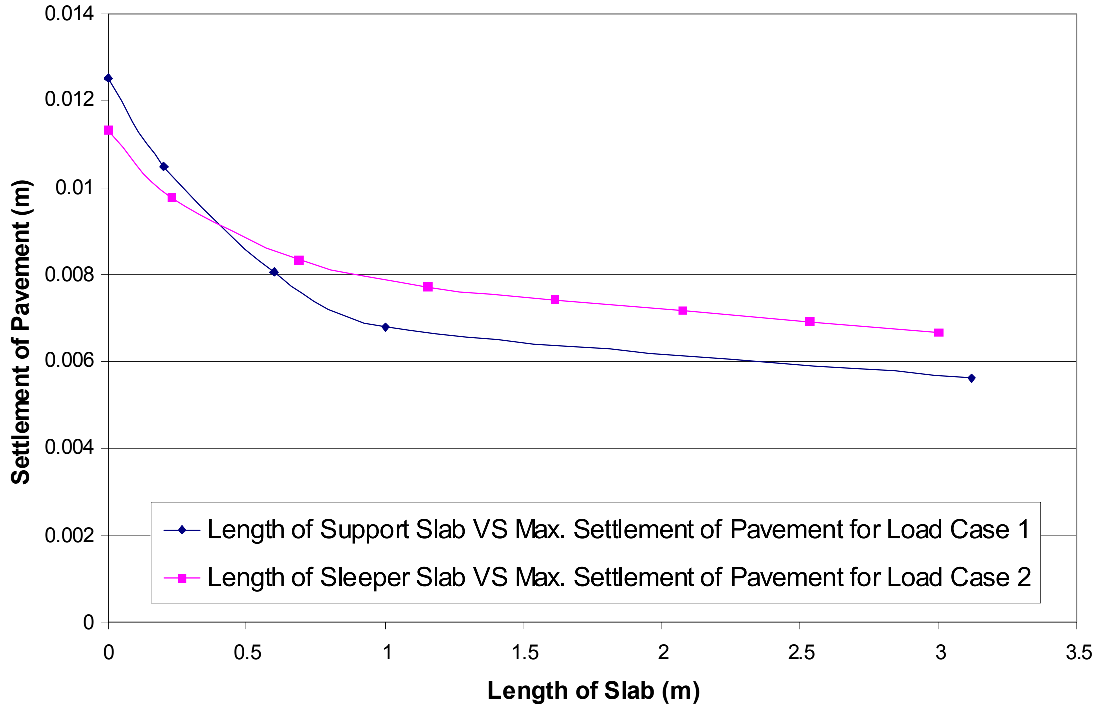


Figure 4.17. Settlements as a Function of the Length of Slab.

Summary of Results

1. The slope of the pavement near the abutment (Figure 4.14) is shown in Table 4.5 for two thicknesses of the abutment wall. The results show that the influence of the thickness of the abutment wall on the bump is limited.

Table 4.5. Gradient of the Differential Settlement on the Approach Slab for the Soft Soil.

Loading Case 4 (Figure 4.4) and Modulus in Zone 3 = 5,000 kPa	0.5 m Wall		1.0 m Wall	
	Δ_1	Δ_2	Δ_1	Δ_2
	-0.84/100	-0.15/100	-0.71/100	-0.25/100

2. The soil stiffness near the abutment (zone 3 in Figure 4.4) affects the slope between the abutment wall and the support slab, and therefore the bump size. If the stiffness is decreased by half, the slope is increased by 20 percent (Figure 4.13). Therefore, a higher stiffness (higher compaction) near the abutment can minimize the bump although the relationship between soil stiffness and bump size is not a linear relationship.
3. The presence of the wall creates a major difference in settlement between the soil right behind the abutment wall and the soil away from the wall. The soil close to the wall is held up by the vertically rigid wall, while the soil away from the wall remains unsupported and settles more. This differential settlement creates a bump. The pavement slope between the abutment wall and the support slab was -0.84/100 with a 0.5 m thickness abutment and 0.13/100 with no abutment wall. These results refer to loading case 4 (Figure 4.4) and a soft soil in zone 3 (Table 4.1).
4. The pavement profiles detailed in the simulations indicate that the transition zone is about 40 ft with 80 percent of the maximum settlement occurring in the first 20 ft for a uniform loading case. Therefore, the bump occurs near the support slab, which is 20 ft away from the bridge abutment.
5. As shown in Figure 4.17, the settlement of the support slabs and the sleeper slab keeps decreasing as the length of both slabs increases. This decrease becomes small when the slabs are over 5 ft. Therefore, the optimum length for both slabs is 5 ft.
6. The high approach embankment (6.4 m) showed 31 percent more settlement of the pavement than the low approach embankment (3 m), and the ratio of settlement is proportional to the ratio of the total height of the model (embankment + natural soil).

CHAPTER 5:
NEW APPROACH SLAB

All the accumulated data indicate that the current bridge approach slab system can lead to a bump. The current system is an articulated double-span approach slab with a significant weakness at the middle hinge (Figure 5.1). This system often experiences a V-shaped dip, which was found at the two test sites. The first section in this chapter describes the current approach slab. The researchers propose two conceptual replacement solutions in the second and third sections.

Current Approach Slab

TxDOT uses a 12-inch-thick approach slab made of reinforced concrete. The approach slab has two 20 ft spans. It is supported by the abutment backwall, the approach backfill, and two slabs: the support slab and the sleeper slab (Figure 5.1). To accommodate the movement of the pavement, a wide flange (WF) steel beam is used on top of the sleeper slab. The pavement side of the wide flange beam can move horizontally and freely in the beam.

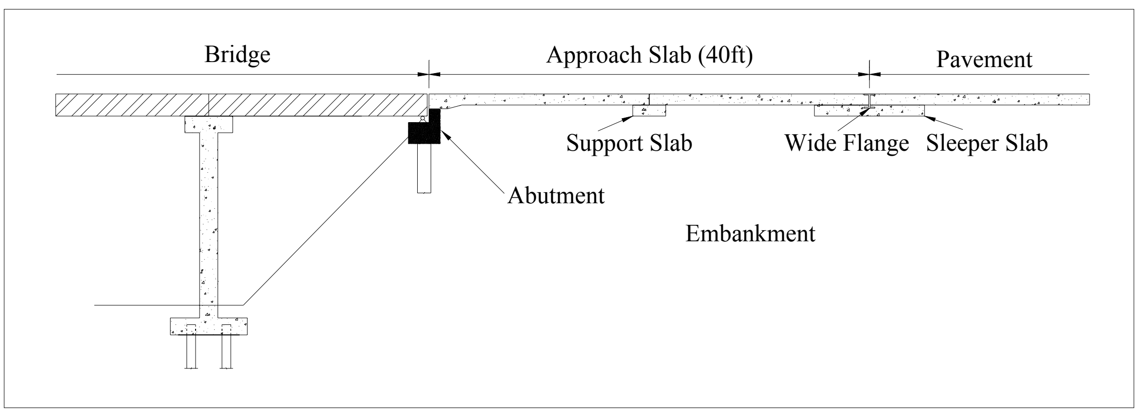


Figure 5.1. Current Approach Slab.

One-Span Approach Slab Designed in Free Span

This solution would consist of a 20-ft-long single slab (possibly ribbed) from the abutment to the sleeper slab (Figure 5.2). It would be designed to carry the full traffic load without support on the soil except at both ends. The current practice is for an approach slab of the same thickness as the adjacent approach pavement, which can likely accommodate a 20-ft free span with support of traffic. The articulation would be removed and the wide flange would be kept on the embankment side as a temperature elongation joint for the pavement. This solution will simplify construction significantly, be less expensive, and place less emphasis on the need for very good compaction close to the abutment wall, which is usually difficult.

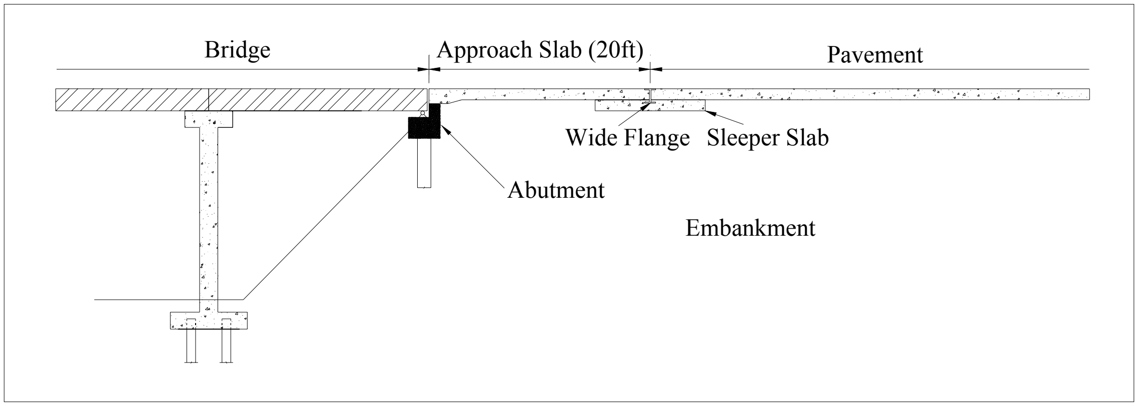


Figure 5.2. One-span Approach Slab.

Abutment on Sleeper Slab

This solution is more bold but it is well worth considering. The approach slab is essentially another span of the bridge. That span rests on deep foundations (most of the time) on the abutment side and on a shallow spread footing on the embankment side (Figure 5.3). This proposed solution of the abutment on the sleeper slab (spread footing) would use the first bridge span as the approach slab and place the abutment on the sleeper slab. This solution requires careful considerations of several issues, but it is a very economical solution that would work very well in principle.

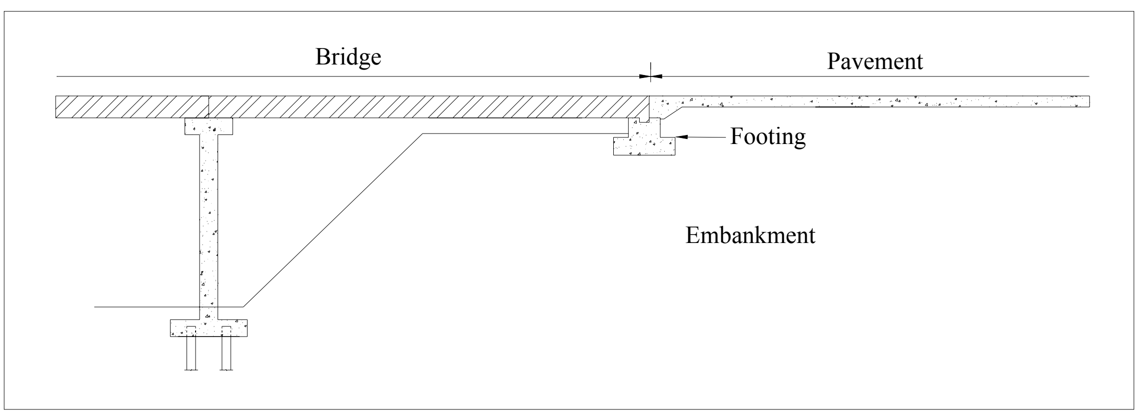


Figure 5.3. Abutment on a Sleeper Slab.

Numerical Simulation for New Approach Slab

A numerical simulation was done for the one-span approach slab. The results for the current approach slab and the one-span approach slab are shown in Figures 5.4 and 5.5, respectively. The maximum settlement and the deformed mesh of those two cases show a little difference. The maximum settlement for the current approach slab is 0.068 m (0.5 m wall, load case 4, and soft soil in zone 3) and 0.071 m for the new approach slab. More detailed results are shown in Appendix G for 0.5 m wall, load case 4, and a soft soil in zone 3.

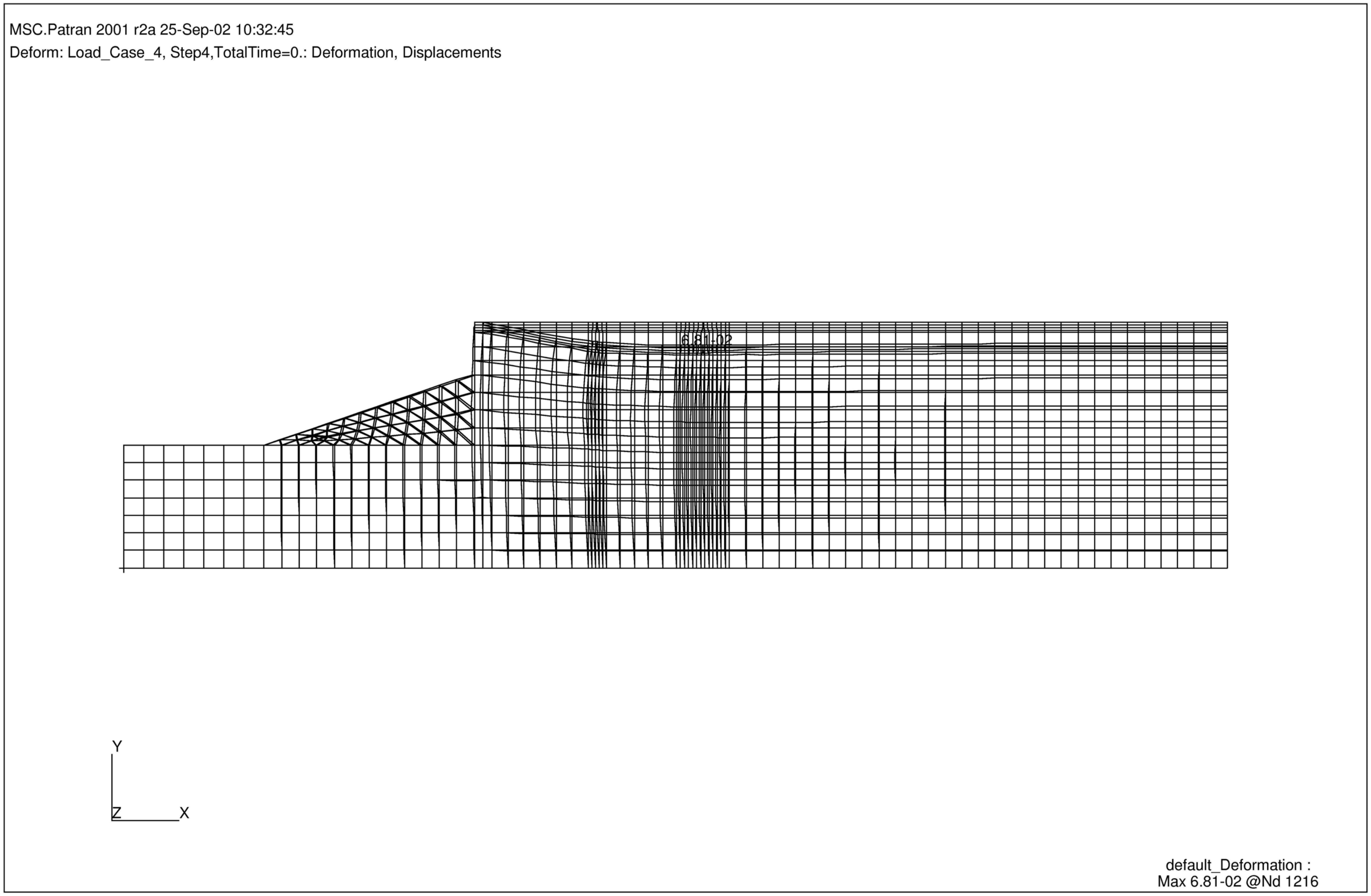


Figure 5.4. Current Approach Slab for a 0.5 m Wall, Load Case 4, and Soft Soil in Zone 3.

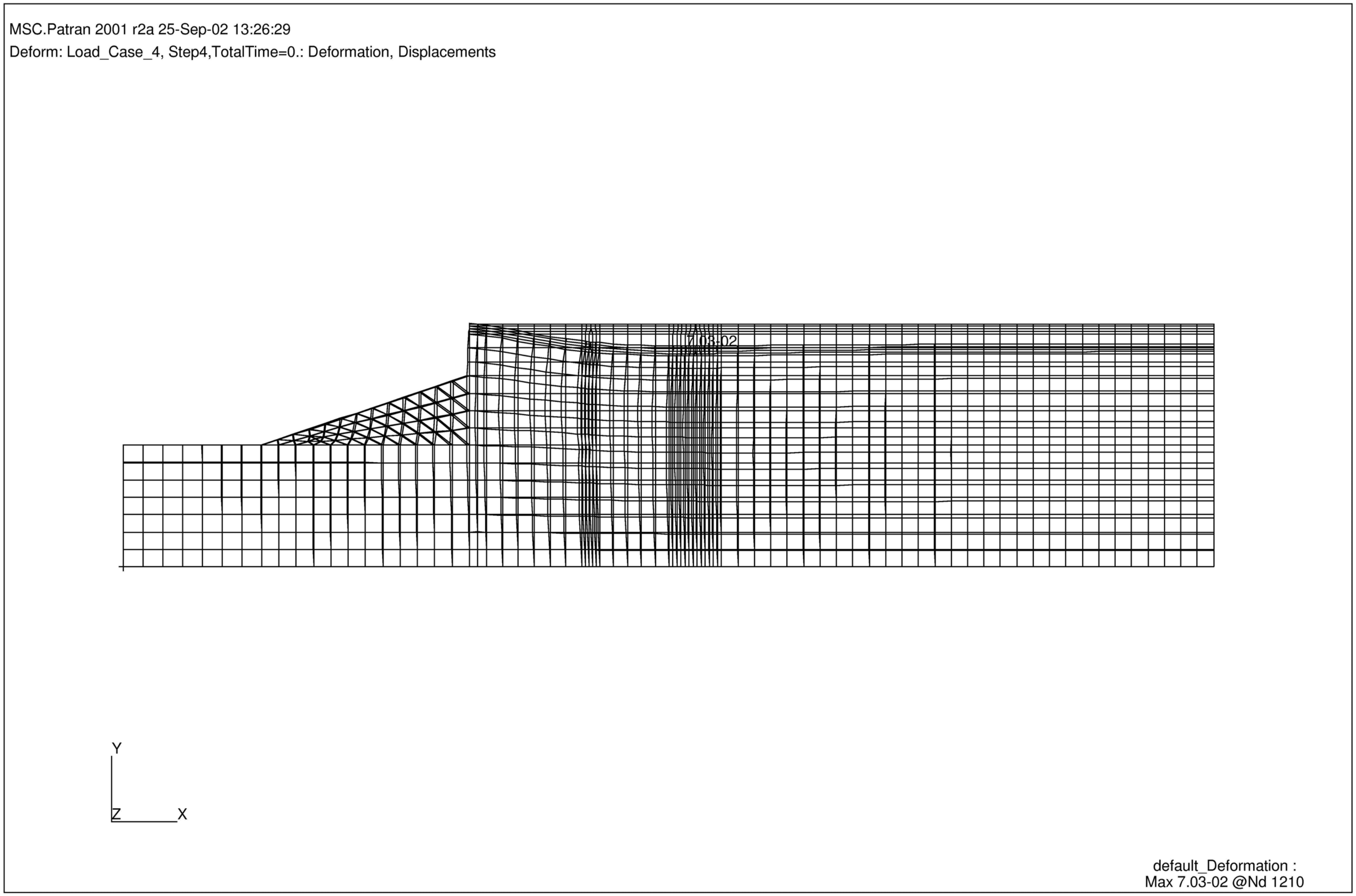


Figure 5.5. One-Span Approach Slab for a 0.5 m Wall, Load Case 4, and Soft Soil in Zone 3.

CHAPTER 6:
MODEL SCALE SIMULATIONS

The BEST device was been designed and built to simulate the bump at the end of the bridge problem. BEST stands for Bridge to Embankment Simulator of Transition. It is a 1/20th scale model of the typical transition. The researchers studied the scaling laws and made decisions on the choice of parameters. One problem was that some parameters scale directly with length (e.g. embankment height), while others do not (e.g. dynamics). An optimum combination of parameters was studied and finally selected. It was chosen to model properly the most important parameters in the system. The soils to fill the container were clay and sand. Filling the BEST device and preparing the experiment took about one week. Running the test for a week generates about 300,000 cycles of loading. Each test, therefore, took two weeks. The purpose of this test is to study the various factors influencing the differential settlement between the embankment and the bridge and to develop alternative solutions for eliminating or minimizing this differential settlement.

The first section of this chapter deals with the dimensional analysis of the problem. The second section describes the BEST device and the soils used. The simulation results are shown in the third section. A summary of results is shown in the fourth section.

Dimensional Analysis

Dimensional analysis is a technique used in physical sciences and engineering to reduce physical properties such as acceleration, viscosity, and energy to their fundamental dimensions of length, mass, and time. This technique facilitates the study of interrelationships of systems (or models of systems) and their properties. Dimensional analysis is often the basis of theoretical and physical models of real situations. Fundamental units (length, time, and either force or mass) are used in analyses. All other quantities such as stress, moment, and velocity are derived from the fundamental units. These units usually come from the fundamental balance laws such as conservation of mass, conservation of energy, and so on.

Buckingham π theory

The Buckingham π theorem states that a function describing a relationship among n quantities, X_i, such as

$$f(X_1, X_2, X_3, \cdots, X_n) = 0 \qquad (6.1)$$

where m primary units are requiring to express the X_i variables can be reduced to the form

$$g(\Pi_1, \Pi_2, \Pi_3, \cdots, \Pi_{n-m}) = 0 \qquad (6.2)$$

where Π_i are nondimensional products of powers of the X_i of the form

$$\Pi_i = X_1^a X_2^b \cdots X_3^c \qquad (6.3)$$

Thus, this very powerful result reduces by the number of primary units, m, the number of variables required to describe the dependent variables.

Application of Dimensional Analysis

The dimensional analysis begins with defining the variables affecting the settlement of the embankment. Figure 6.1 and Table 6.1 show the variables and their dimensions.

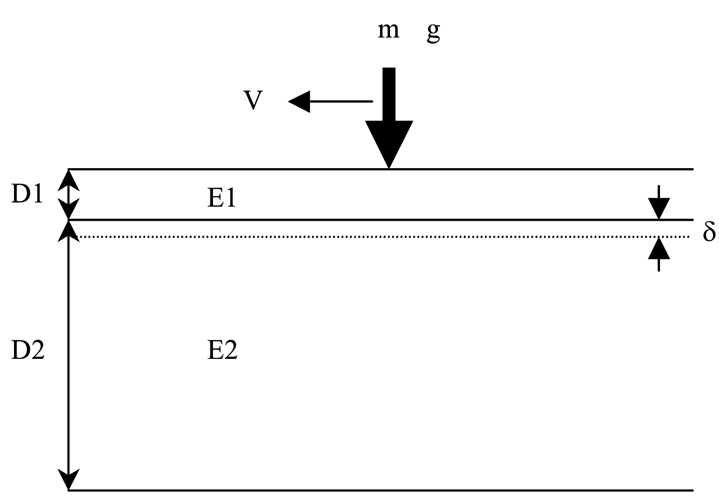


Figure 6.1. Variables for Dimensional Anaylsis.

Table 6.1. Parameters and Dimensions.

Quantity	Parameters	Dimension
Settlement	δ	L
Mass	M	FT^2/L
Gravity	G	L/T^2
Pavement Property	$E_1 \times I_1$	$F\text{-}L^2$
Pavement Depth	D_1	L
Soil Young's Modulus	E_2	F/L^2
Soil Depth	D_2	L
Velocity	V	L/T
Acceleration	A	L/T^2

After defining the variables, grouping according to the fundamental units such as force group (F group), time group (T group), and length group (L group) is performed as shown in Table 6.2. All the variables should be placed in three groups (F group, L group, and T group) with a dimension, and then one variable is selected from each group as a repeating variable. The dependent variable, in this case settlement (δ) , can not be the repeating variable. The selection of repeating variable depends on experience, but any of them will work. In this study, the researchers selected the mass (m), the pavement depth (D_1), and the gravity (g) for repeating variables.

Table 6.2. Fundamental Units.

Group	Variables	Repeating Variable
F Group	m, $E_1 \times I_1$, E_2	M
L Group	D_1, D_2, δ	D_1
T Group	g, V, a	G

The product of power of repeating variables and each nonrepeating variable in terms of dimensions as shown in Equation (6.4) become 1 for this product to be dimensionless (Equation (6.5)). Equations (6.4) to (6.10) show one example of the calculation procedure and the result.

$$\Pi_1 = m^a \cdot g^b \cdot D_1^{\ c} \cdot (E_1 \cdot I_1)^d \tag{6.4}$$

$$\Pi_1 = m^a \cdot g^b \cdot D_1^{\ c} \cdot (E_1 \cdot I_1)^d \Rightarrow (\frac{FT^2}{L})^a (\frac{L}{T^2})^b (L)^c (F - L^2)^d = 1 \tag{6.5}$$

$$F^{a} \cdot F^{d} \Rightarrow a + d = 0 \tag{6.6}$$

$$L^{-a} \cdot L^{b} \cdot L^{c} \cdot L^{sd} \Rightarrow -a + b + c + 2d = 0 \tag{6.7}$$

$$T^{2a} \cdot T^{-2b} \Rightarrow 2a - 2b = 0 \tag{6.8}$$

$$if \;\; a = 1 \; then \; d = -1, b = 1, \; and \;\; c = 2 \tag{6.9}$$

$$\Pi_1 = \frac{m \cdot g \cdot D_1^{\;2}}{E_1 \times I_1} \tag{6.10}$$

The dimensions for the model can be determined from Equation (6.11).

$$(\Pi_1)_{prototype} = \left[\frac{m \cdot g \cdot D_1^{\;2}}{E_1 \times I_1}\right]_{prototype} = (\Pi_1)_{model} = \left[\frac{m \cdot g \cdot D_1^{\;2}}{E_1 \times I_1}\right]_{model} \tag{6.11}$$

In the same manner, Equations (6.12) to (6.16) were obtained and used.

$$\Pi_2 = \frac{m \cdot g}{D_1^{\;2} \cdot E_2} \tag{6.12}$$

$$\Pi_3 = \frac{D_2}{D_1} \tag{6.13}$$

$$\Pi_4 = \frac{\delta}{D_1} \tag{6.14}$$

$$\Pi_5 = \frac{V^2}{g \cdot D_1} \tag{6.15}$$

$$\Pi_6 = \frac{a}{g} \tag{6.16}$$

Based on these relationships, the results of the dimensional analysis for a model scaled $1/20^{th}$ of the length are presented in Table 6.3. The actual variables being used in the field are represented in the prototype column (Field). For a perfect model simulation, the parameters should be scaled directly in the model (Target) values, but this is not always possible and the model (Actual) values were used throughout the BEST test for practical reasons.

Table 6.3. Dimensional Analysis Results.

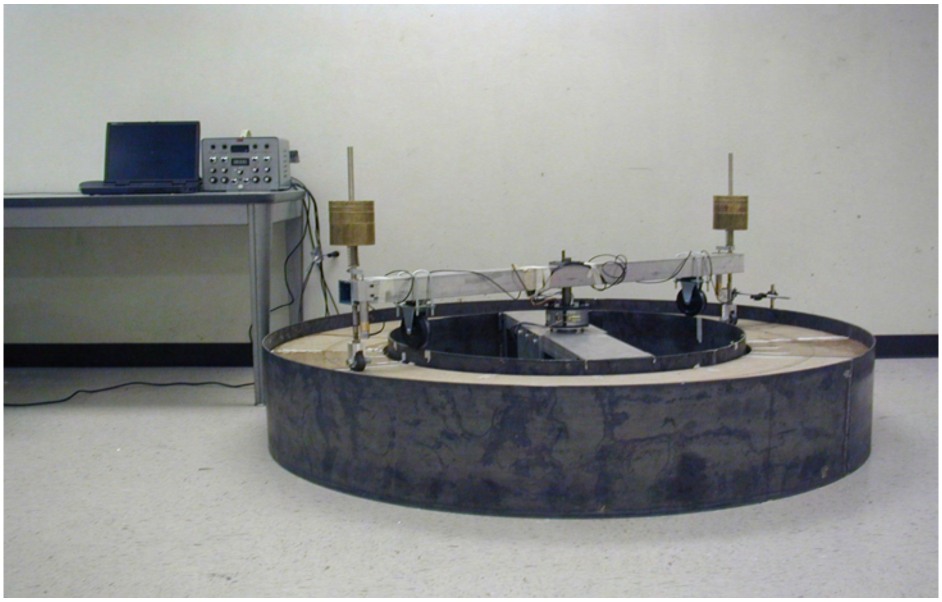
Quantity	Symbol	Prototype (Field)	Model (Target)	Model (Actual)
Settlement (m)	δ	0.05	0.0025	0.0012-0.005
Mass (kg)	m	10,000	20	8
Gravity (m/sec^2)	g	9.8	9.8	9.8
Pavement Elastic Modulus (Pa)	E_1	30×10^9	10×10^9	10×10^9
Moment of Inertia (m^4)	I_1	8.7×10^{-3}	8.87×10^{-8}	1.71×10^{-7}
Pavement Property (N-m^2)	$E_1\times I_1$	1.73×10^8	8.87×10^2	1.71×10^3
Pavement Depth (m)	D_1	0.3	0.015	0.019
Soil Young's Modulus (MPa)	E_2	28	23	10.0
Soil Depth (m)	D_2	5.19	0.26	0.26
Velocity (km/h)	V	112	25	6.9
Acceleration (m/sec^2)	a	20-40	20-40	15-30

BEST Device and Soil

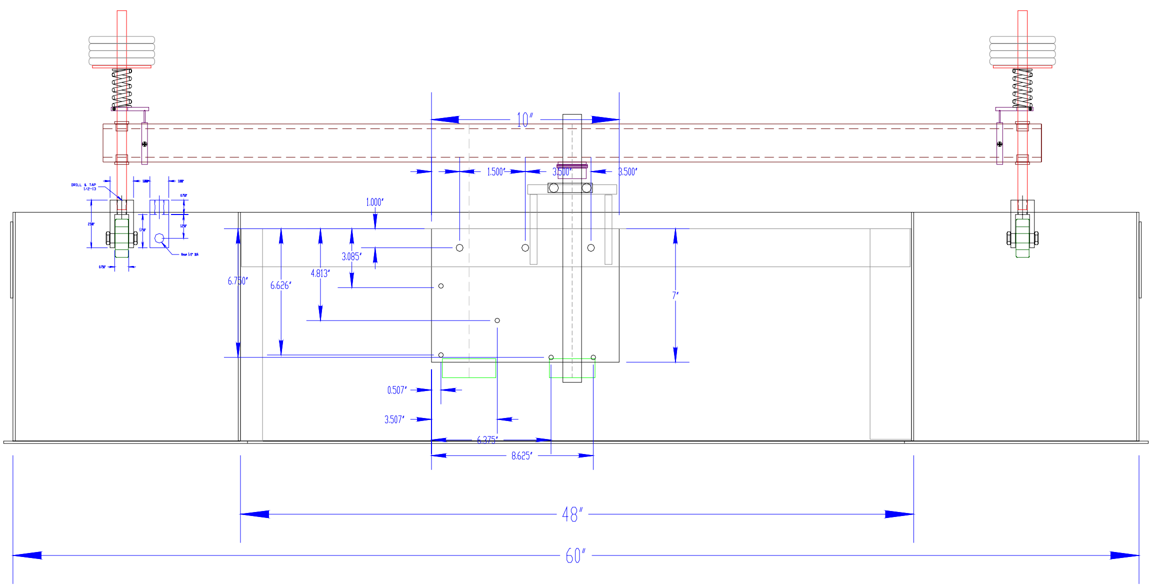
BEST Device

The BEST device was constructed to carry out model tests on the approach slab, bridge, and pavement assembly. It consists of a laboratory-scale driven wheel guided around a circular track by a rotating arm as shown in Figure 6.2. A motor in the center of the tank runs the wheel at various speeds. The wheel passes over the embankment, approach slab, and bridge once during each cycle around the track. The data obtained during a test are the elevations of the riding surface as a function of time and cycles.

Shackel and Arora (1978) and Road Transport Research (1985) gave a description of many of the test tracks developed for pavement studies. Almost all of these test tracks can either be classified as linear or circular tracks. Linear tracks have a test wheel move forward and forth. Circular tracks have a rotating arm carrying a test wheel that runs around a circular test pavement or track containing the test section (Barenberg and Hazarida, 1976; Paterson, 1972).

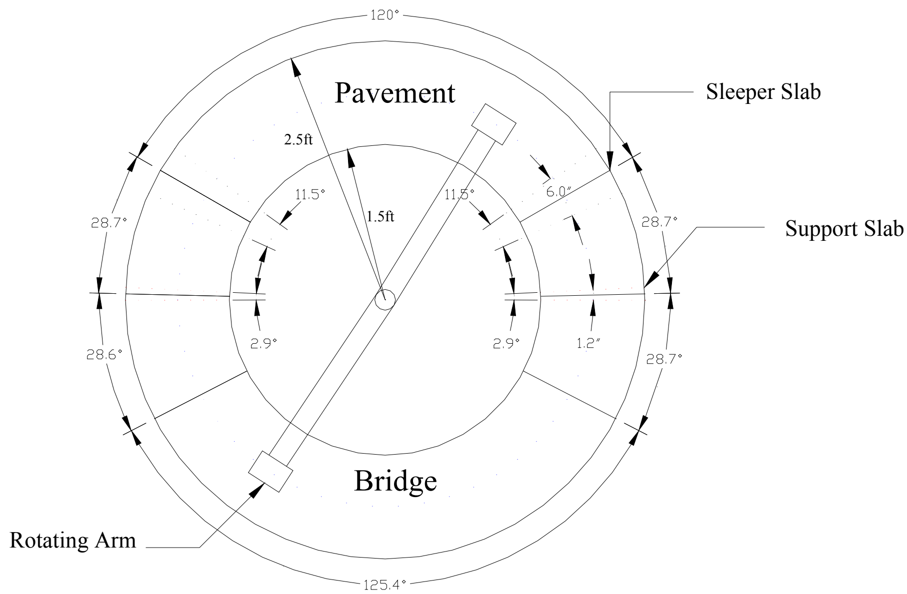


(a) Photo of BEST Device



(b) Cross Section of BEST Device

Figure 6.2. BEST Device.



(c) Plan View of BEST Device

Figure 6.2. BEST Device (cont.).

Setup of the BEST Device

Sand and porcelain clay were used to simulate the embankment in the BEST tests. The setup procedure is shown in Figures 6.3 to 6.5. Sand was placed in the tank except at the bridge sections, which were supported by columns on the floor of the device (Figure 6.3). The compaction was done by using a hand tamper with an area 10 inch by 10 inch and weighing 10 lbs. Each test for the sand has three layers. To keep the density of the sand consistent throughout the tests, 9 blows/ft^2/layer for the high level of compaction effort, and 3 blows/ft^2/layer for the low level of compaction effort at the approach slab sections which are 2 ft away from each end of the bridge, were used. The pavement section as shown in Figure 6.2 was compacted 9 blows/ft^2/layer. For the clay case, the porcelain clay blocks were placed at the approach slab sections as shown in Figure 6.4 and then the gaps between the clay blocks were filled and leveled with sand. Figure 6.5 shows finished setup for the BEST test. The finished height of the embankment was about 10 inches. The pavement was made of ¾ -inch plywood and simply placed over the embankment.

Figure 6.3. Setup for Sand and Compaction.

Figure 6.4. Setup for Clay.

Figure 6.5 Finished Setup before Placing the Pavement and Approach Slab on the Embankment.

Velocity

The velocity of the rotating arm is V_0 (1 cycle/2 seconds, 6.89 km/hr) with an 8 kg weight on the top of the wheel. Velocities equal to 0.4 V_0 and 2 V_0 are also available by changing the gears. Figure 6.6 shows the rotating arm at a speed of V_0.

Figure 6.6. Rotating Arm.

Loading and Measurement

The loading carriage consists of a loading system with a wheel and a driving unit (see Figure 6.2). The tire is 1/20th the size of a full-scale truck tire and is connected to a rod that slides up and down freely through the rotating arm. A spring is placed between the rotating arm and the weight to simulate the suspension system. A weight of up to 8 kg is placed on the spring to simulate the vehicle weight.

To monitor the acceleration of the weight, an accelerometer is fitted on top of the weight. An analog to digital signal converter is used to transmit the data from the linear variable differential transformer (LVDT) to a laptop computer. Figure 6.7 shows the measuring system. When the elevation of the roadway is to be measured, the test with the wheel is interrupted, the cart shown in Figure 6.7 is placed, and the elevation is recorded with respect to the sides of the device through the use of an LVDT placed on the wheel.

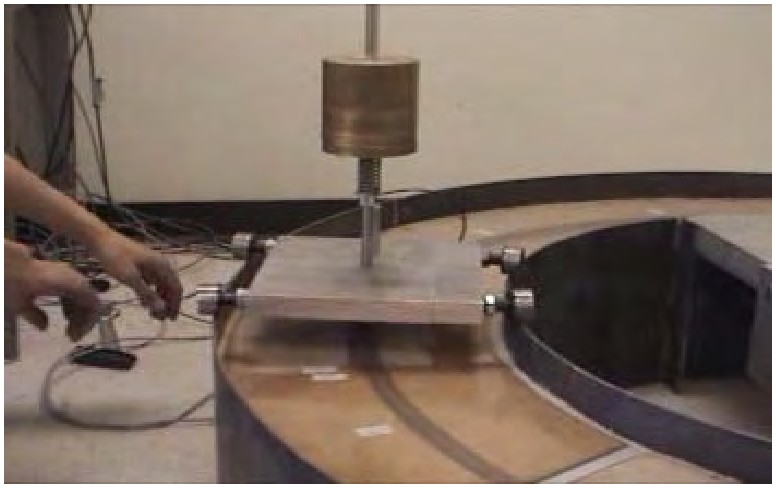


Figure 6.7. Elevation Measuring System.

Test Plan

The research team performed ten tests to evaluate the effectiveness of the one-span approach slab. Four tests are for the current (two-span) approach slab configuration and six tests are for the proposed one-span approach slab. Table 6.4 shows the test matrix.

Table 6.4. Combination of Tests.

Test No.	Approach Slab	Soil	Compactio n Level	Mass (kg)	Velocity (km/h)
1	One-Span	Sand	High	8	6.89
2	One-Span	Sand	Low	8	6.89
3	One-Span	Clay	-	8	6.89
4	One-Span	Sand	Low	8	6.89
5	One-Span	Sand	Low	8	2.76, 6.89, and 13.78
6	One-Span	Sand	Low	1	6.89
7	Current	Sand	High	8	6.89
8	Current	Sand	Low	8	6.89
9	Current	Clay	-	8	6.89
10	Current	Clay	-	8	2.76, 6.89, and 13.78

Soil

Sand and clay were used for this test. Basic soil tests were done for these two soils to determine the soil properties. Table 6.5, Figure 6.8, and Figure 6.9 show dry unit weights, compaction test results, and moduli results from triaxial tests for the sand. To measure the unit weight and its water content, a consolidation ring that is 1 ½ inch in diameter and 1 inch thick was pushed into the sand after finishing the compaction of the sand. After that pushing, the consolidation ring was carefully taken out with the sand by placing a thin plate at the bottom of the ring. The unit weight and the water content were measured using the cored sand sample, and the dry unit weight was then calculated.

Figure 6.10 shows sieve analysis for the clay. The consolidation ring was used to measure the unit weight and the water content of the clay. Sampling was done before the test by pushing the consolidation ring into the clay block and taking it out with clay. The dry unit weight (Table 6.6) was calculated from the measured unit weight and the water content. Table 6.7 presents other basic test results for the clay, including moduli values obtained from unconfined compression tests. The detailed results are shown in Appendix H.

Table 6.5. Dry Unit Weight of Sand.

Test No.	**1**	**2**	**4**	**5**	**6**	**7**	**8**
ω (%)	4.67	7.17	4.92	4.89	4.90	5.00	7.03
γ_d (pcf)	102.0	85.4	84.2	85.7	86.6	101.2	85.6

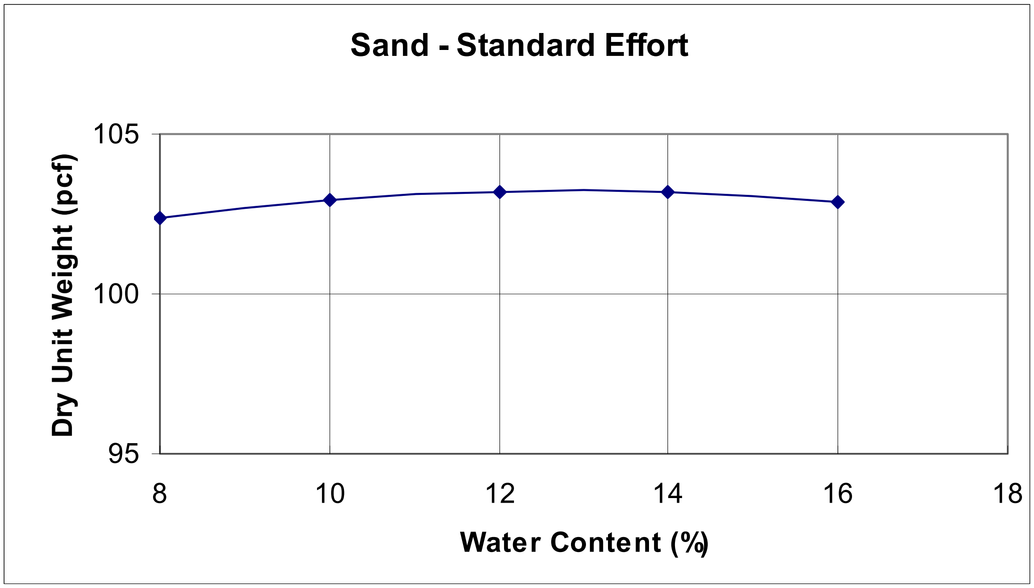


Figure 6.8. Compaction Test for Sand.

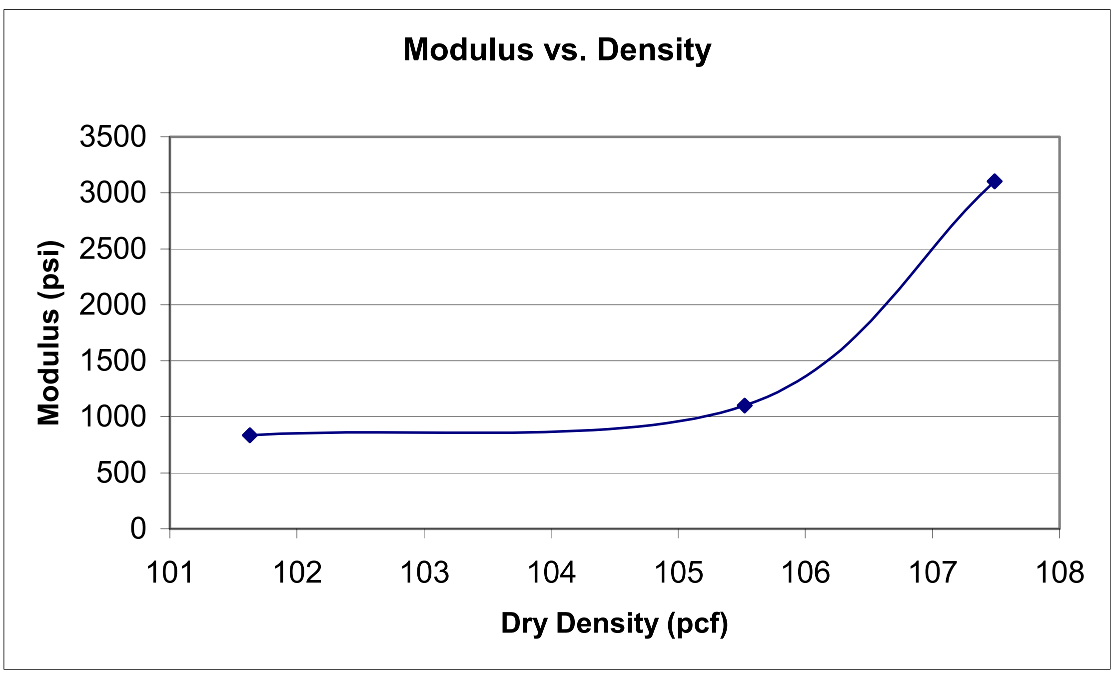


Figure 6.9. Modulus of Sand.

Table 6.6. Dry Unit Weight of Clay.

Sample No.	1	2	3	4
ω (%)	26.8	26.5	26.1	26.4
γ_d (pcf)	90.87	96.36	98.04	94.97

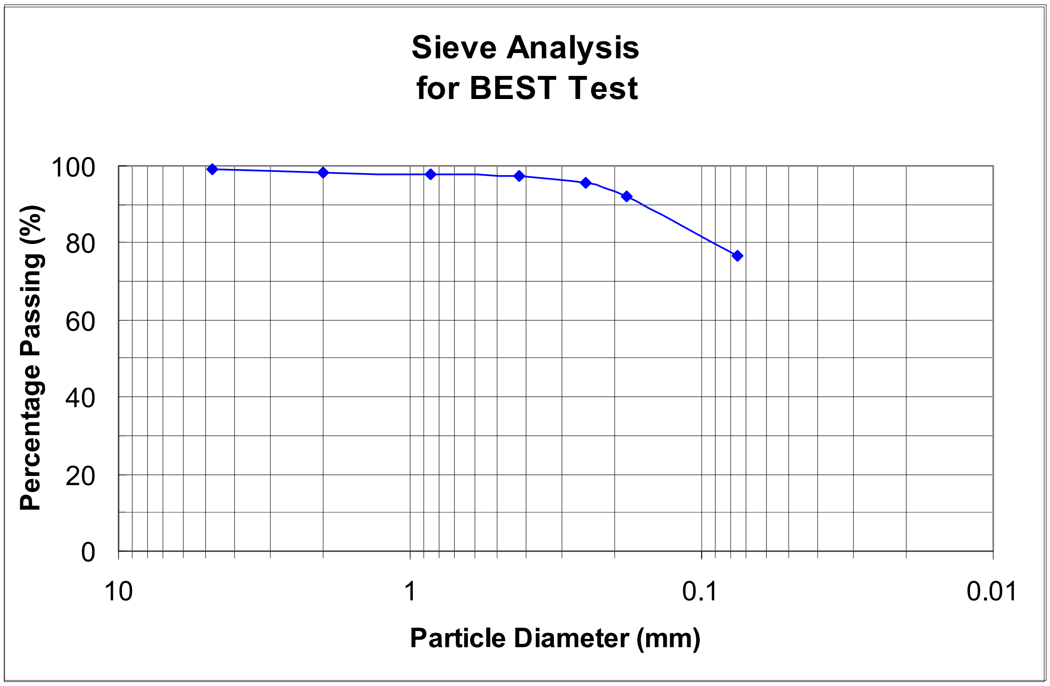


Figure 6.10. Sieve Analysis Result of Clay.

Table 6.7. Basic Test Results of Clay.

Sample No.	Liquid Limit (%)	Plastic Limit (%)	Plastic Index	Young's modulus (E_{25}) (psi)
2	34.44	18.29	16.15	165
3	34.56	18.54	16.02	146
4	34.23	18.10	16.13	119

Simulation Results

Ten tests were done as shown in Table 6.4. Different conditions were used to evaluate the bump at the end of the bridge. The accelerometer gave the acceleration in flight for each measured cycle. The settlement at designated points was measured using the measuring system shown in Figure 6.7. The repeatability of this measurement was about 0.0005 inch or 0.00127 mm.

Test 1

This test was done with a one-span approach slab, sand, a high compaction effort, and 200,000 cycles. The results are shown in Figures 6.11 and 6.12. All the test data are shown in Appendix I.

Test 2

This test was done with a one-span approach slab, sand, low compaction efforts and 200,000 cycles. The results are shown in Figures 6.13 and 6.14. All the test data are shown in Appendix I.

Test 3

This test was done with a one-span approach slab, clay, and 200,000 cycles. The results are shown in Figures 6.15 and 6.16. Acceleration results detained with the accelerometer on the ridging mass are shown in Figures 6.17 and 6.18. All the test data are shown in Appendix I.

Test 4

This test was done to check the repeatability of the BEST device. This test is the same as Test No. 2. The results are shown in Figures 6.19 and 6.20. The comparison with Test No. 2 is very good and verifies the repeatability of the BEST tests. All the test data are shown in Appendix I.

Test 5

This test was done with a one-span approach slab, sand, low compaction effort, and various velocities (0.4 V_o, V_o, and 2 V_o), and 500,000 cycles. The results are shown in Figures 6.21 and 6.22. The three curves in Figure 6.22 represent the settlement of the sleeper slab at the beginning of bridge, at the end of the bridge, and the average of those two. All the test data are shown in Appendix I.

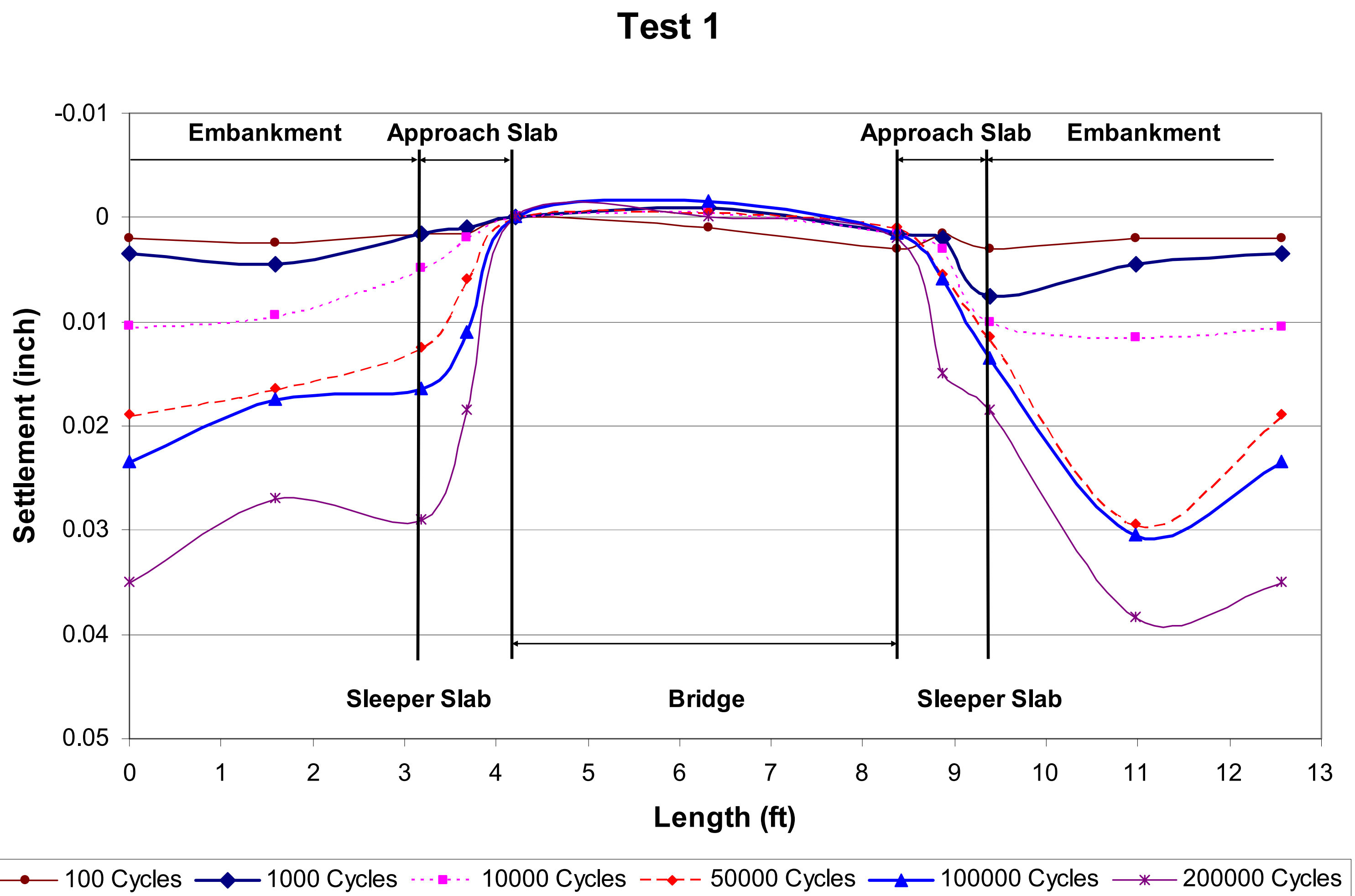


Figure 6.11. Total Profile for Test No. 1.

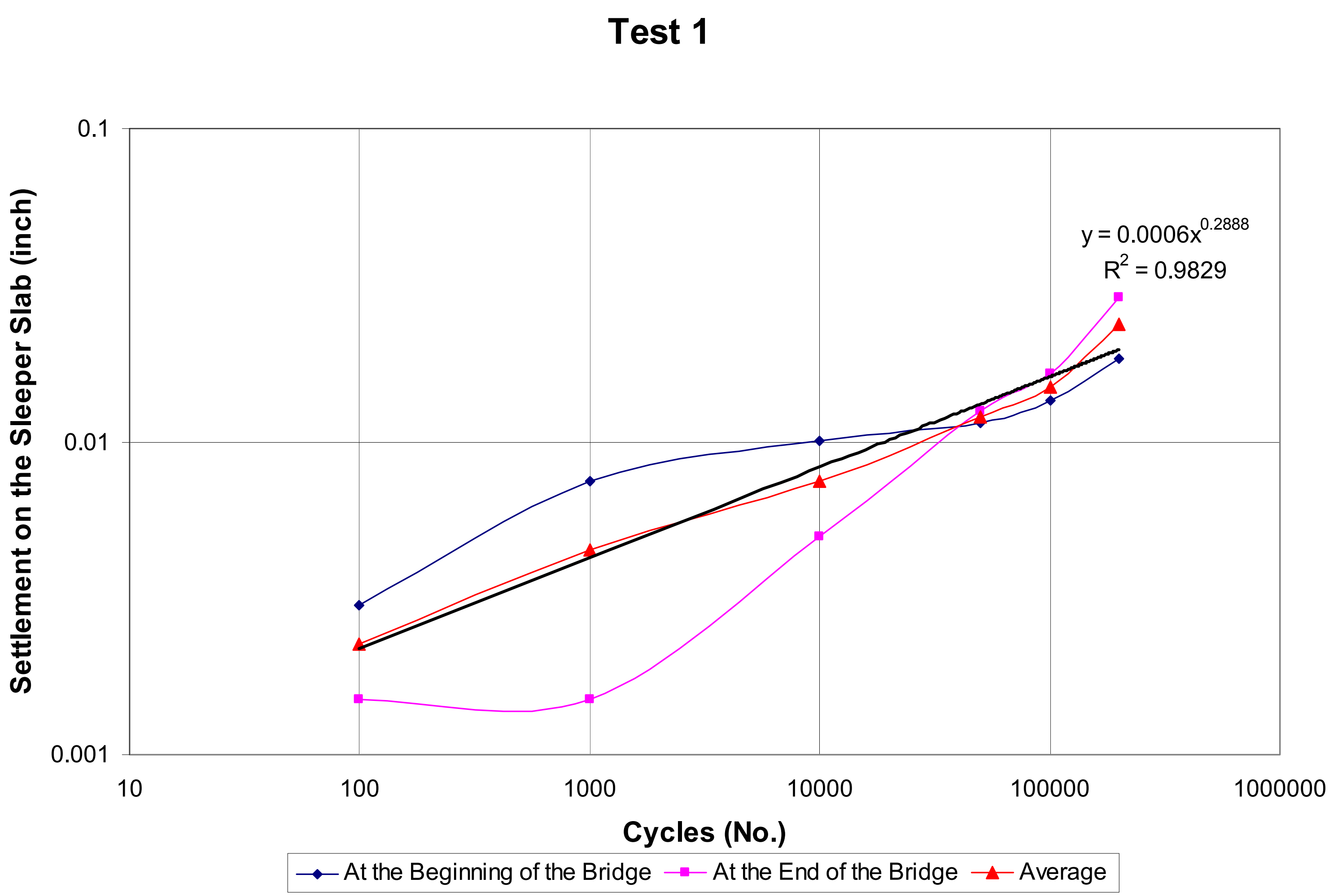


Figure 6.12. Settlement of the Sleeper Slab at Different Cycles for Test No. 1.

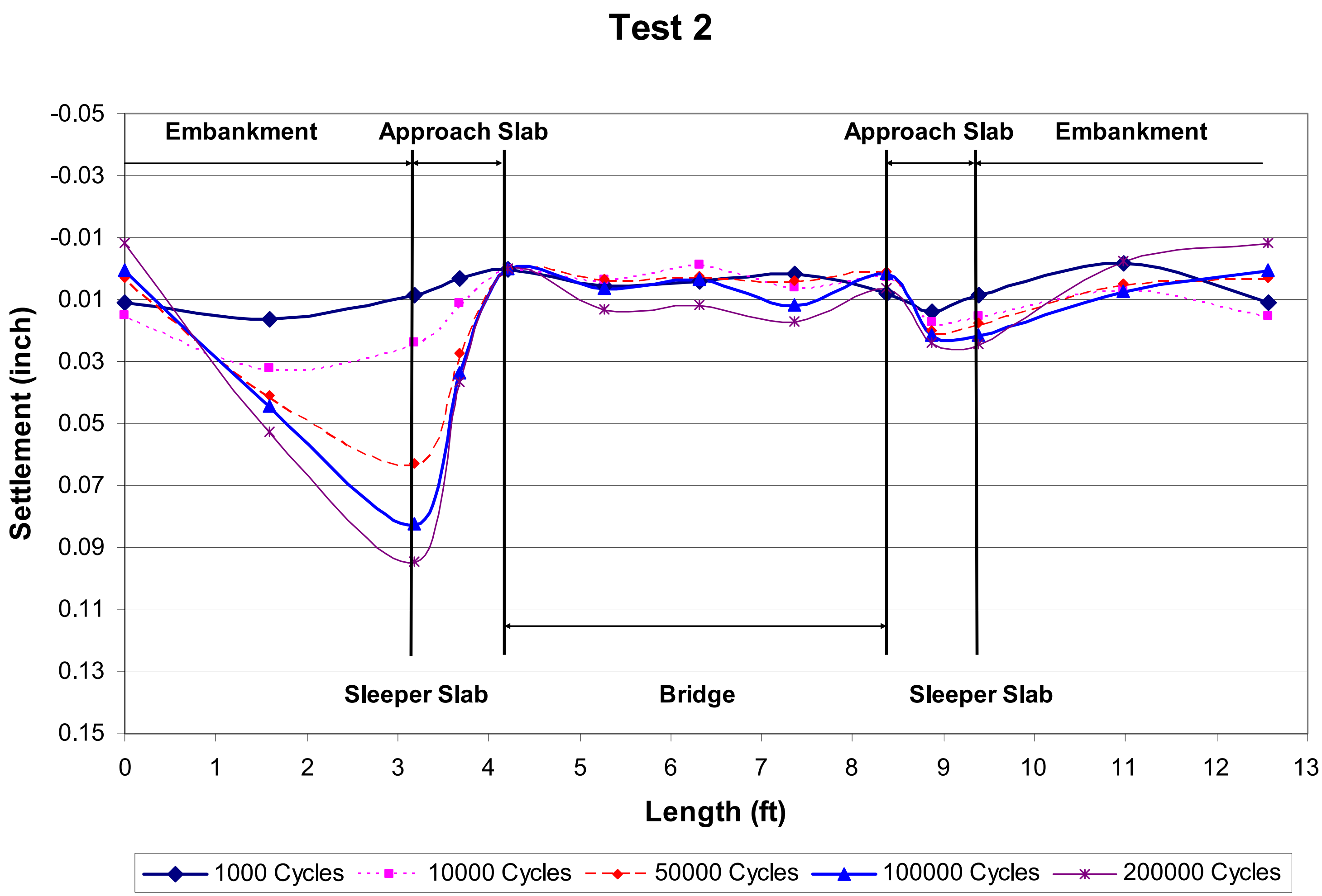


Figure 6.13. Total Profile for Test No. 2.

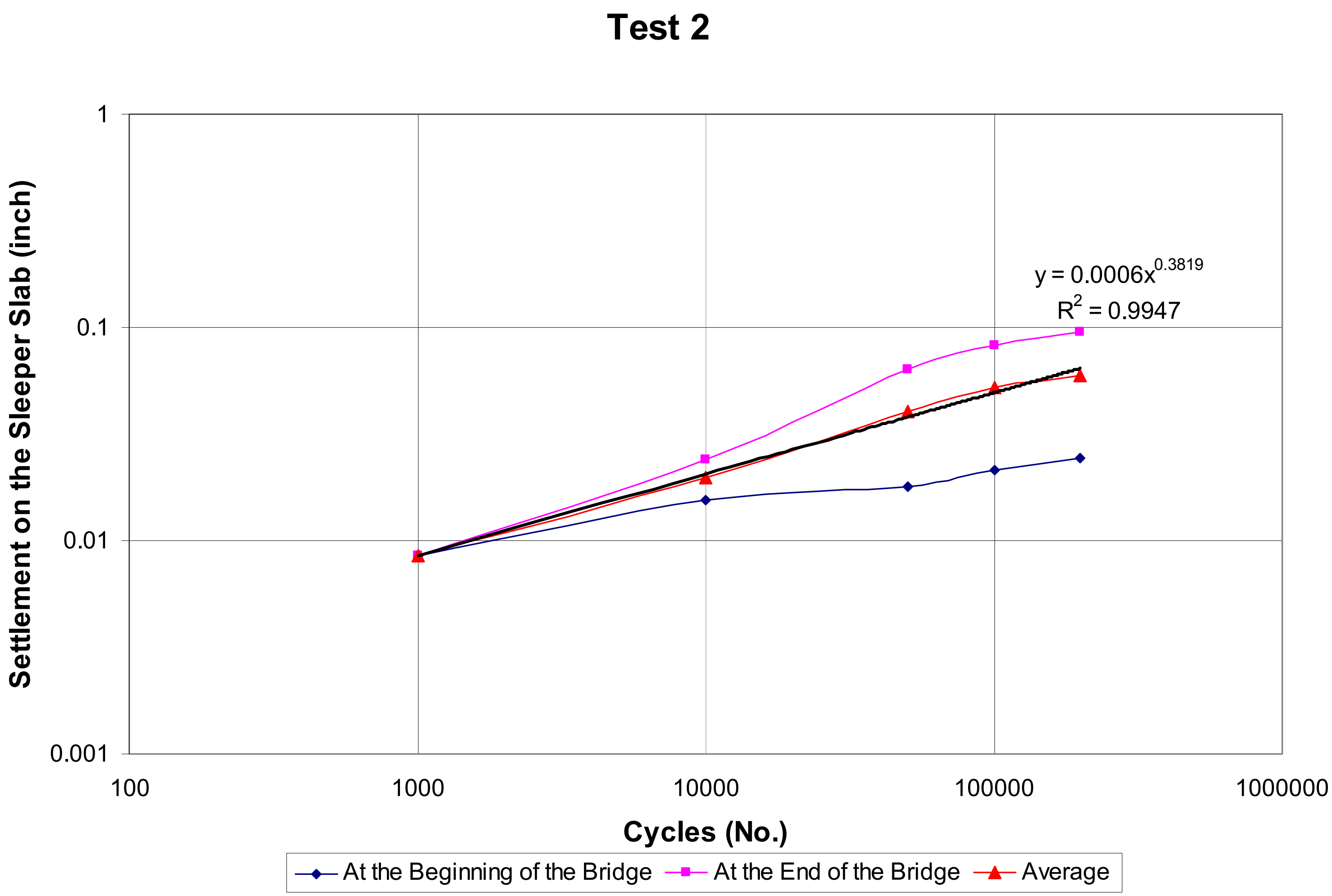


Figure 6.14. Settlement of the Sleeper Slab at Different Cycles for Test No. 2.

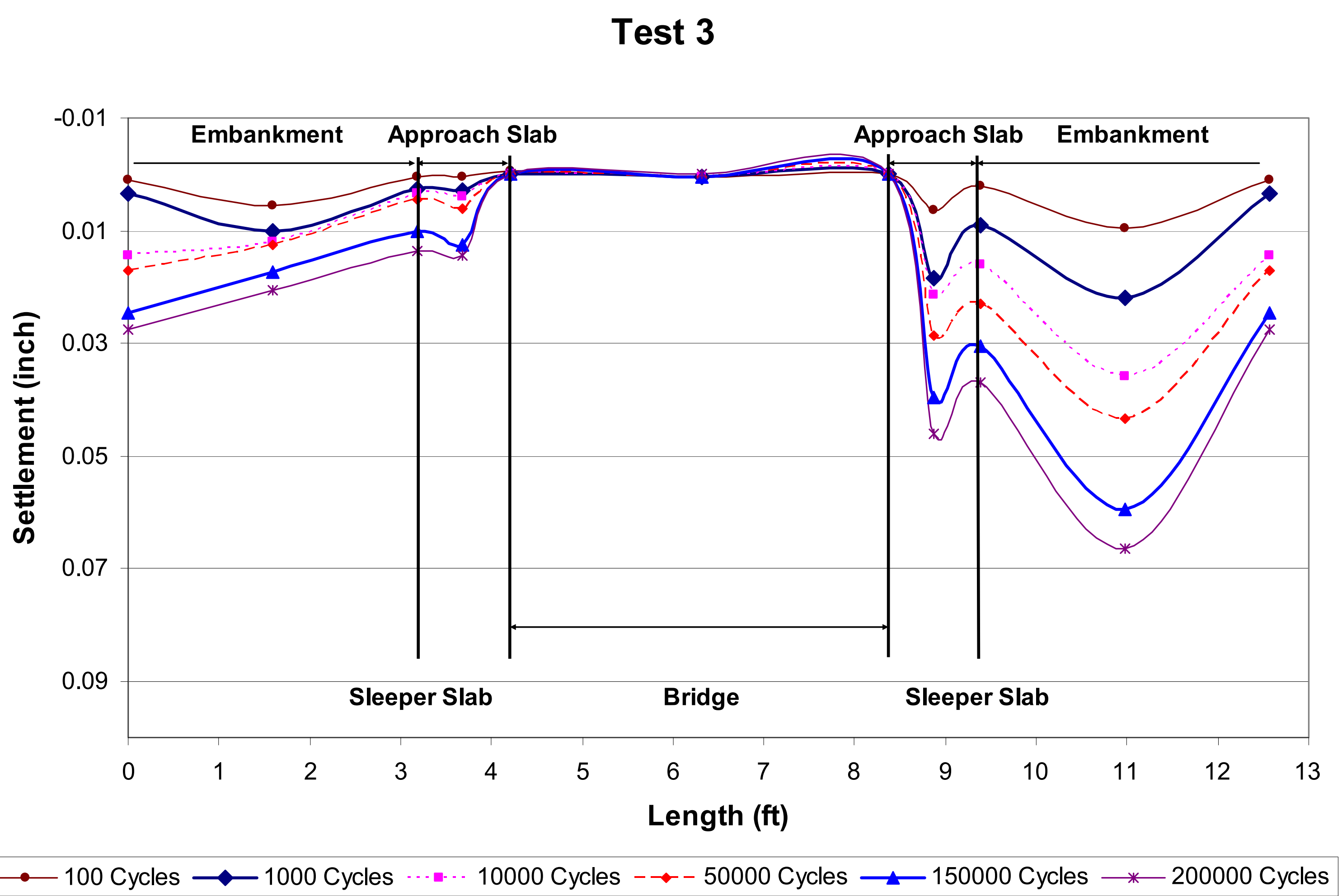


Figure 6.15. Total Profile for Test No. 3.

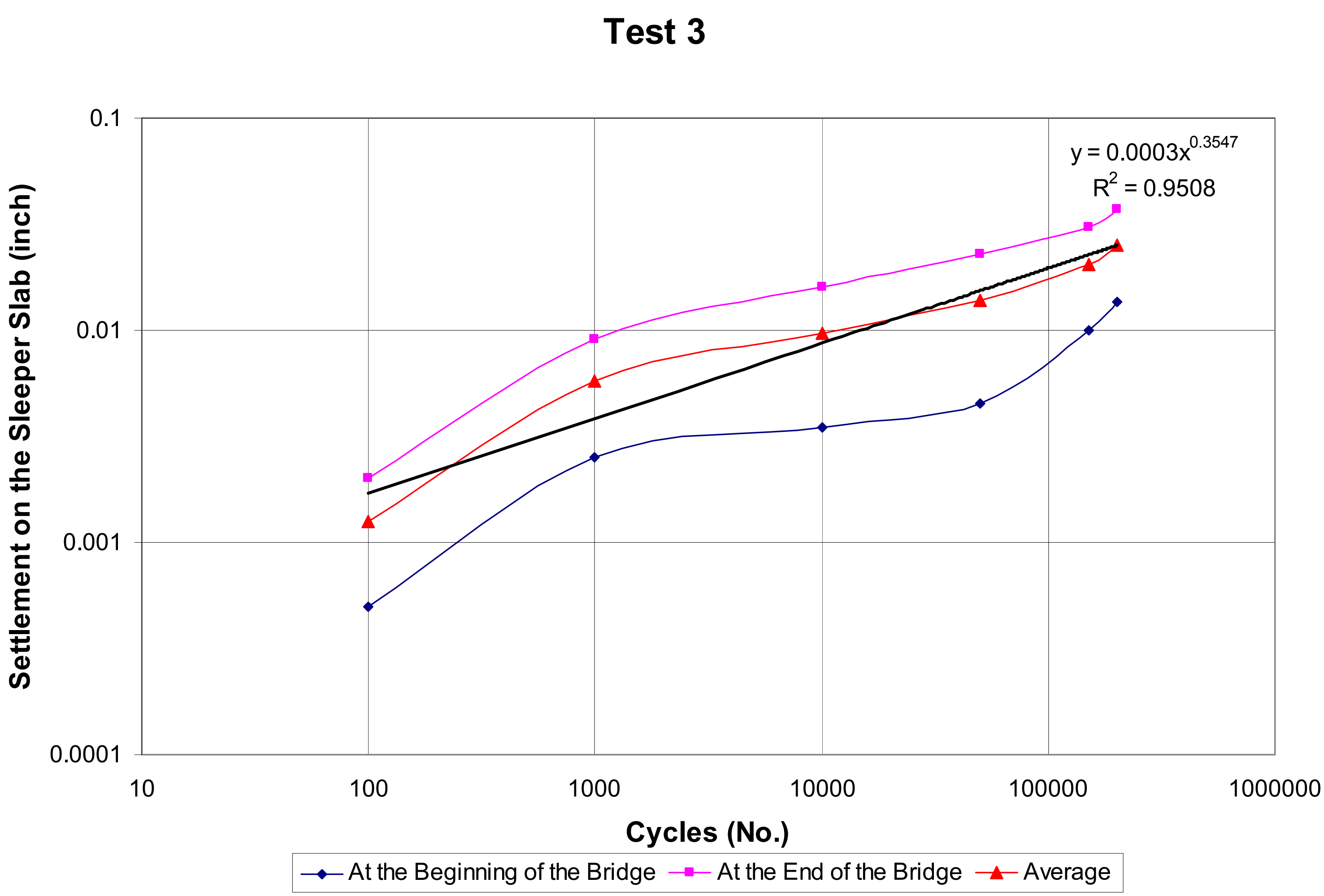


Figure 6.16. Settlement of the Sleeper Slab at Different Cycles for Test No. 3.

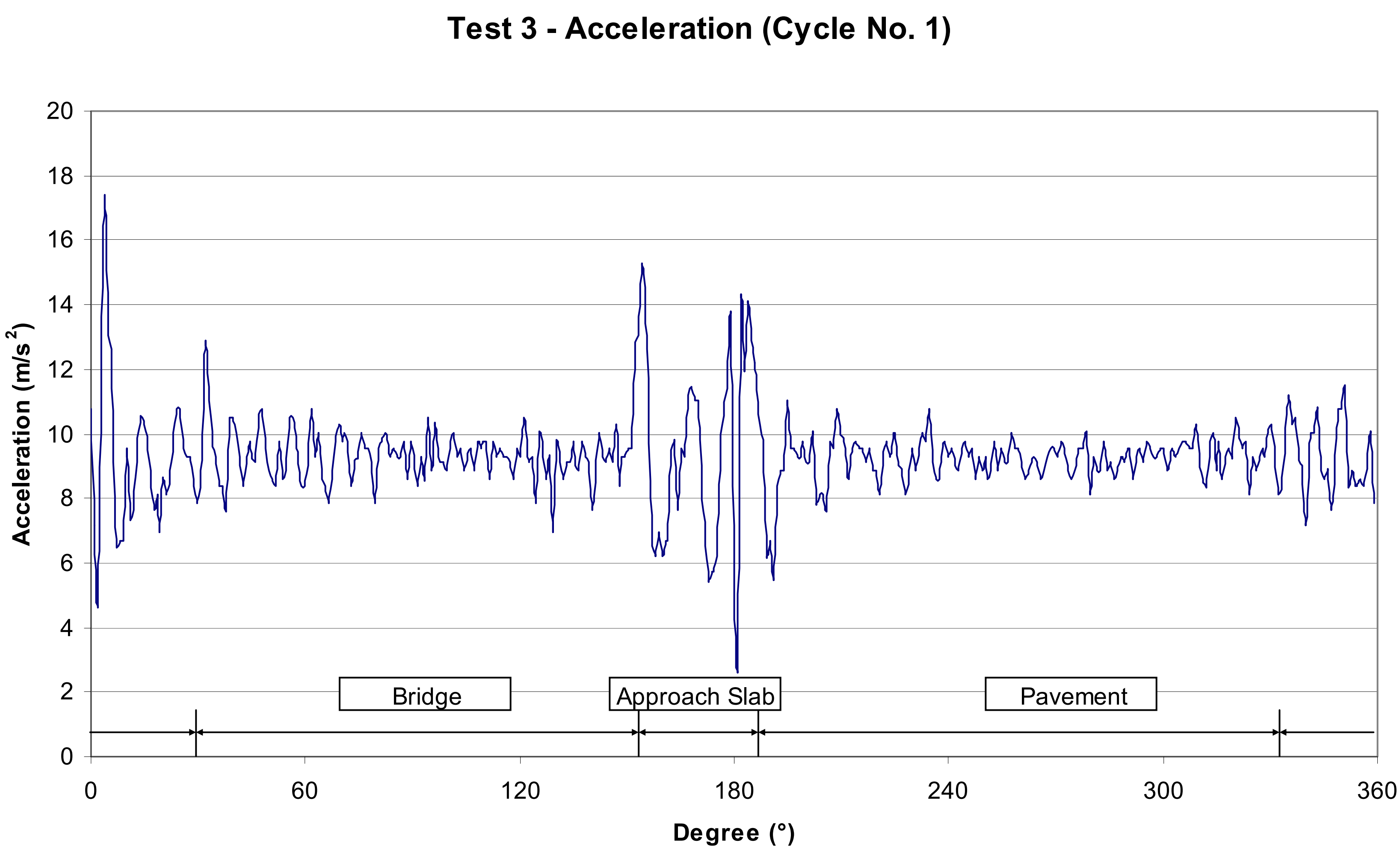


Figure 6.17. Acceleration of the Weight in Test No. 3 at Cycle No. 1.

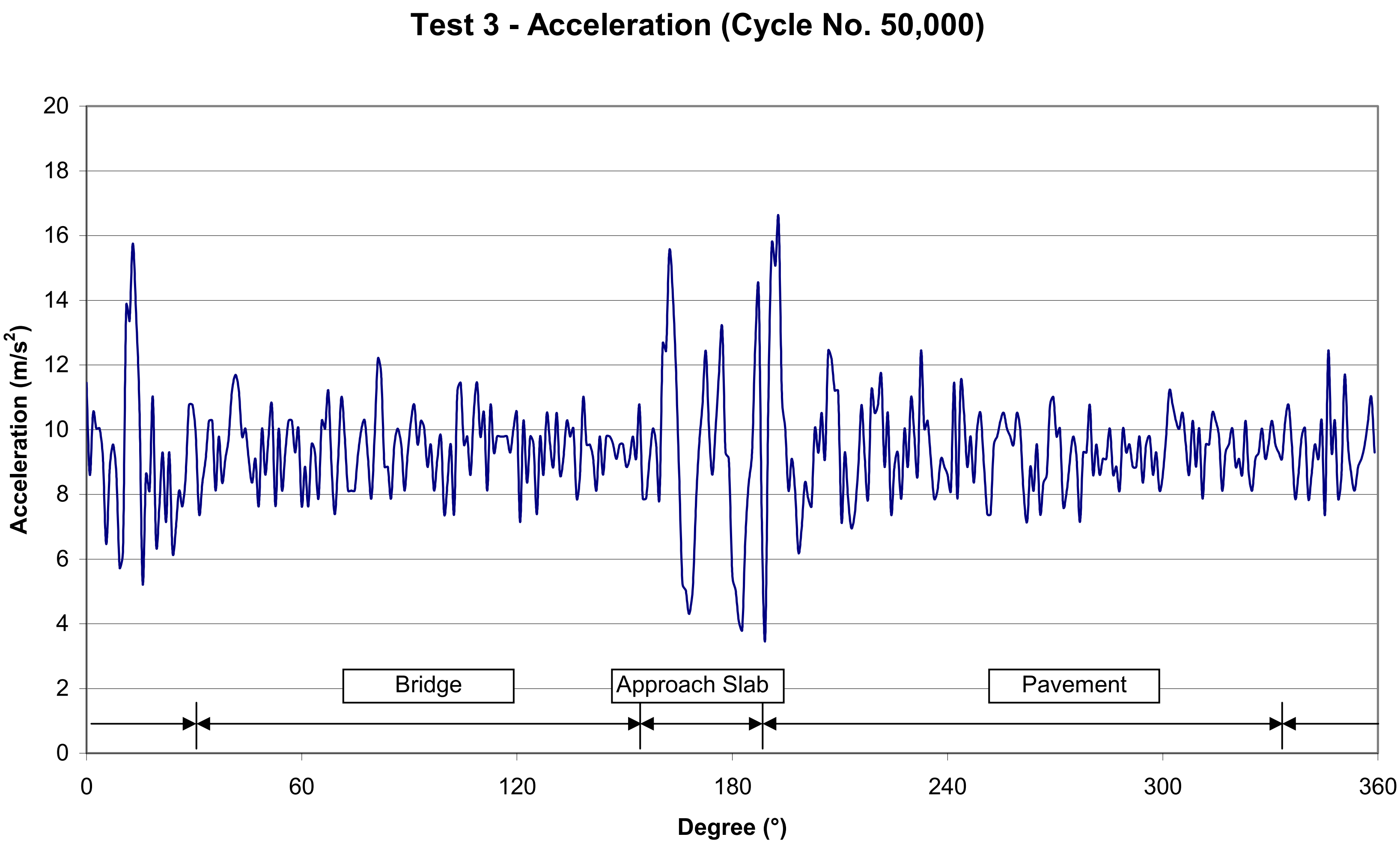


Figure 6.18. Acceleration of the Weight in Test No. 3 at Cycle No. 50,000.

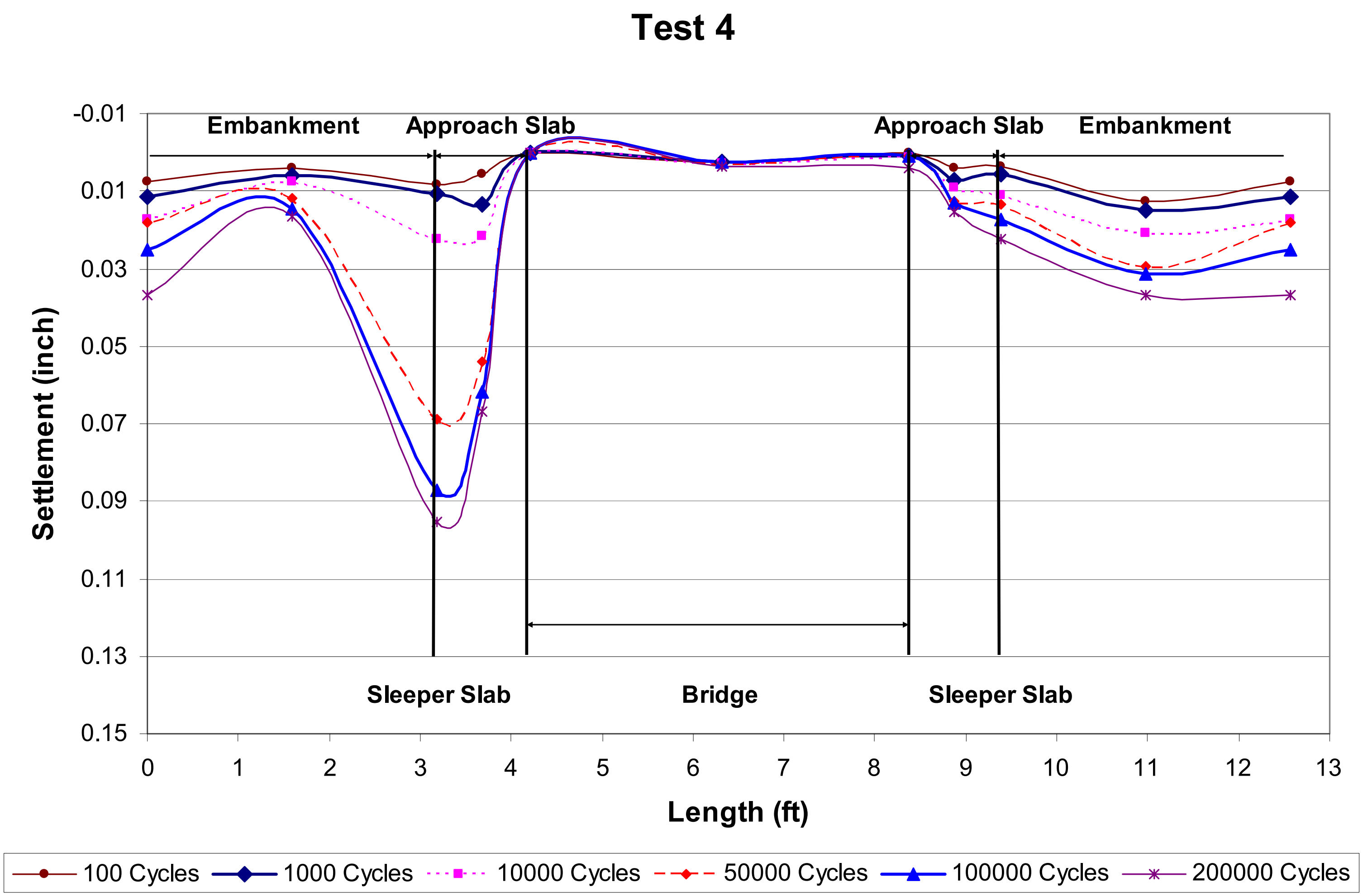


Figure 6.19. Total Profile for Test No. 4.

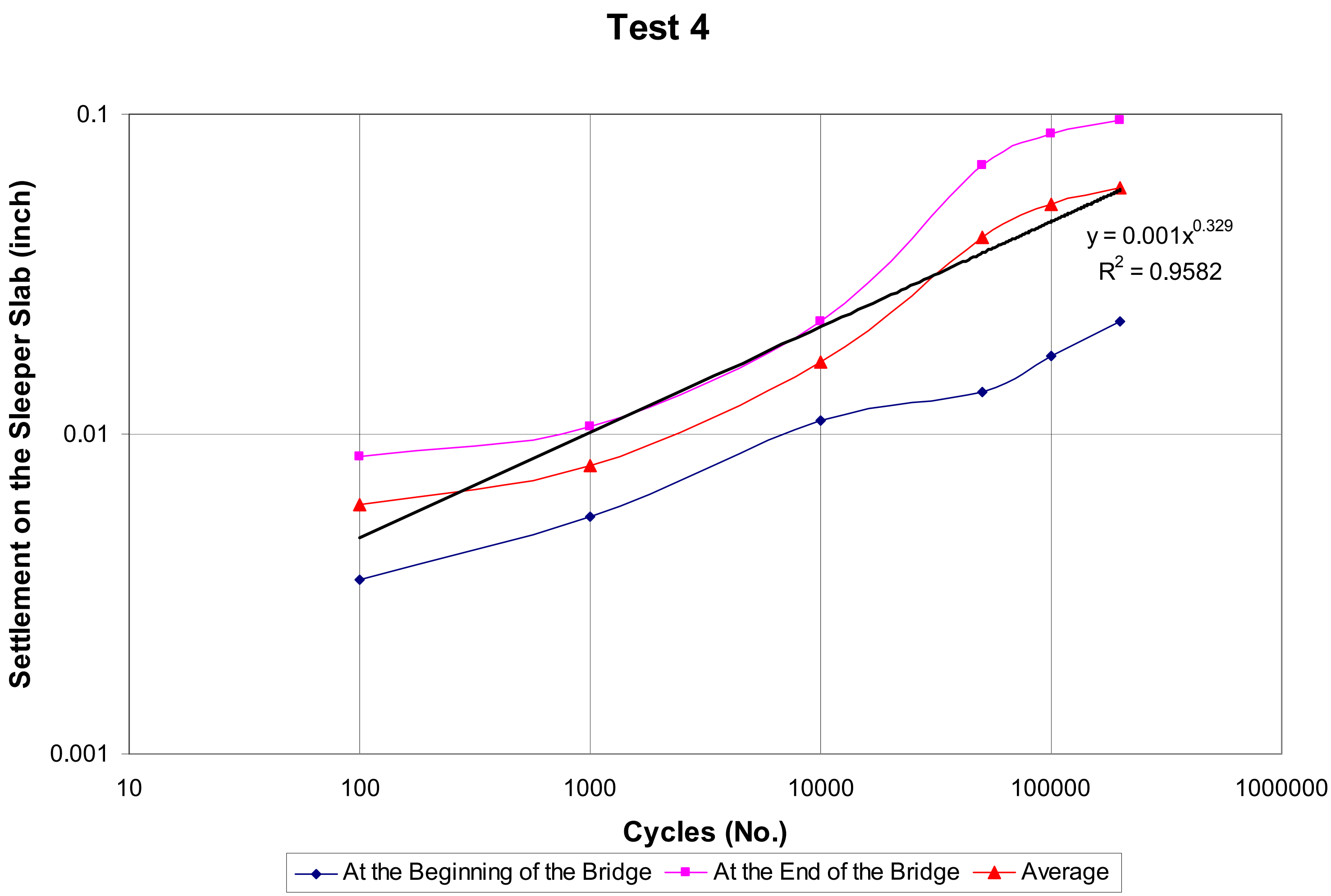


Figure 6.20. Settlement of the Sleeper Slab at Different Cycles for Test No. 4.

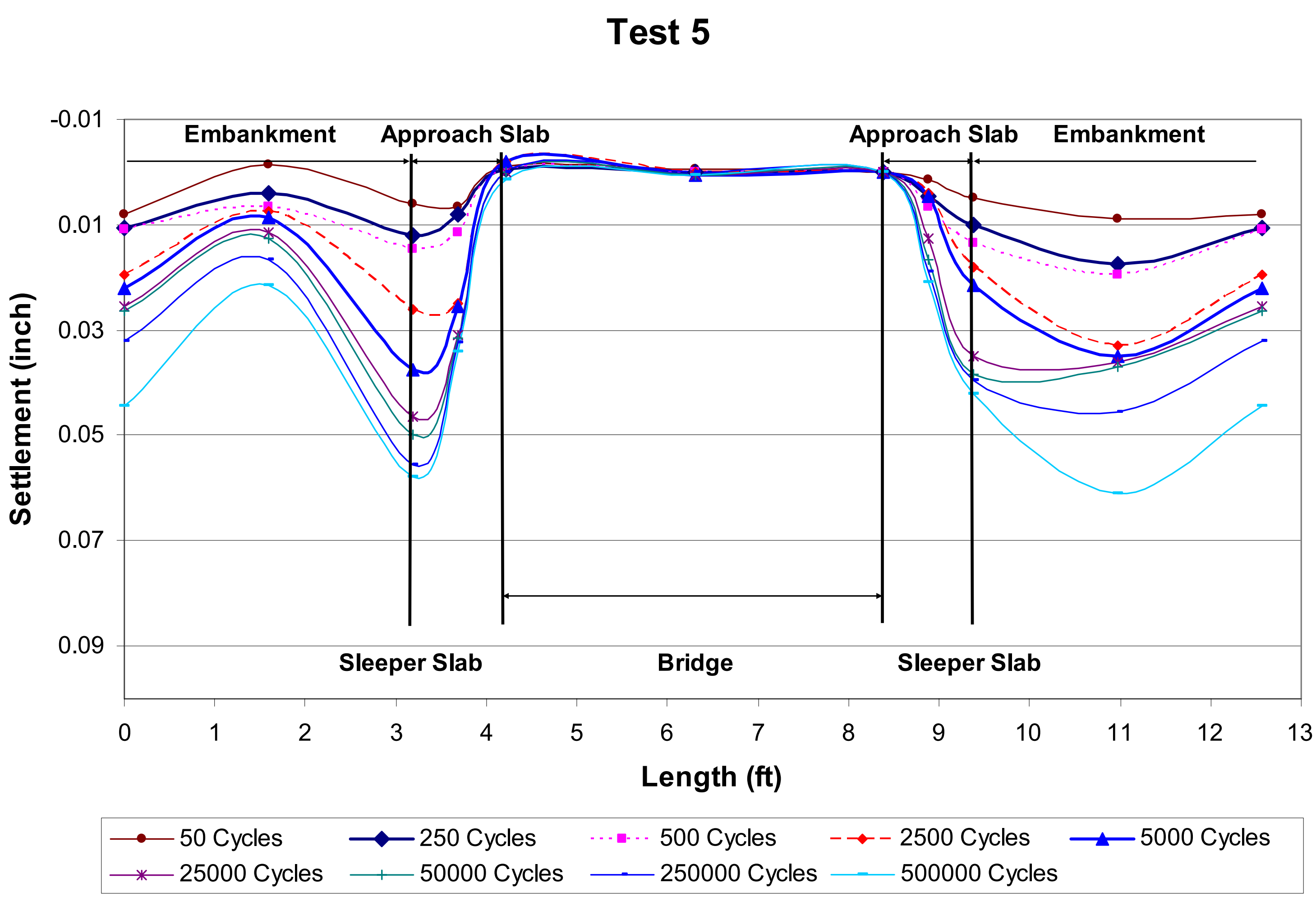


Figure 6.21. Total Profile for Test No. 5.

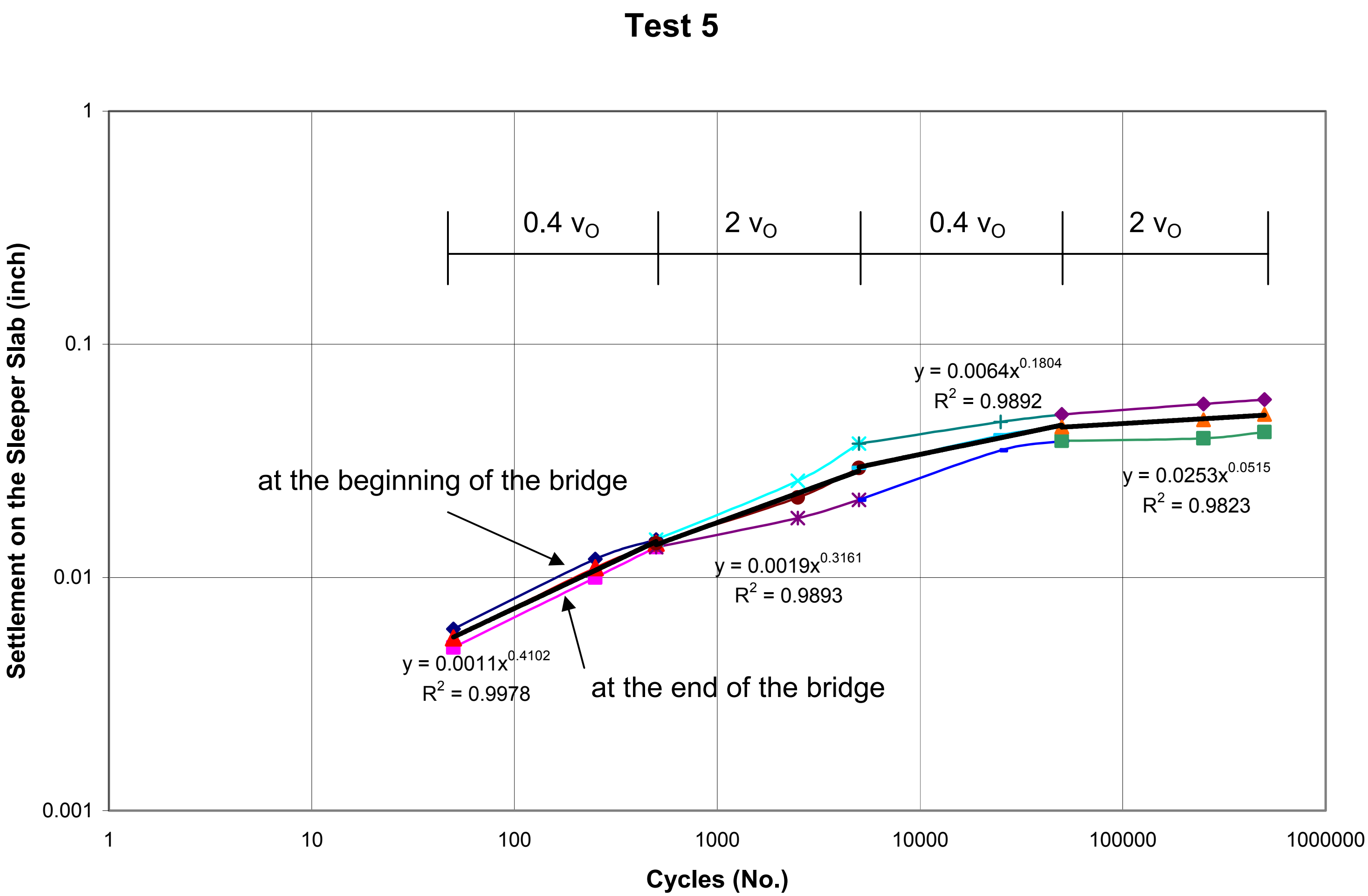


Figure 6.22. Settlement of the Sleeper Slab at Different Cycles for Test No. 5.

Test 6

This test was done with a one-span approach slab, sand, low compaction effort, a smaller weight on the wheel, and 200,000 cycles. The results are shown in Figures 6.23 and 6.24. All the test data are shown in Appendix I.

Test 7

This test was done with the current approach slab, sand, high compaction effort, and 400,000 cycles. The results are shown in Figures 6.25 and 6.26. All the test data are shown in Appendix I.

Test 8

This test was done with the current approach slab, sand, low compaction effort, and 200,000 cycles. The results are shown in Figures 6.27 and 6.28. All the test data are shown in Appendix I.

Test 9

This test was done with the current approach slab, clay, and 200,000 cycles. The results are shown in Figures 6.29 and 6.30. Acceleration results detained with the accelerometer on the riding mass are shown in Figures 6.31 and 6.32. All the test data are shown in Appendix I.

Test 10

This test was done with the current approach slab, clay, and 100,000 cycles. The results are shown in Figures 6.33 and 6.34. Acceleration results are shown in Figures 6.35 and 6.36. All the test data are shown in Appendix I.

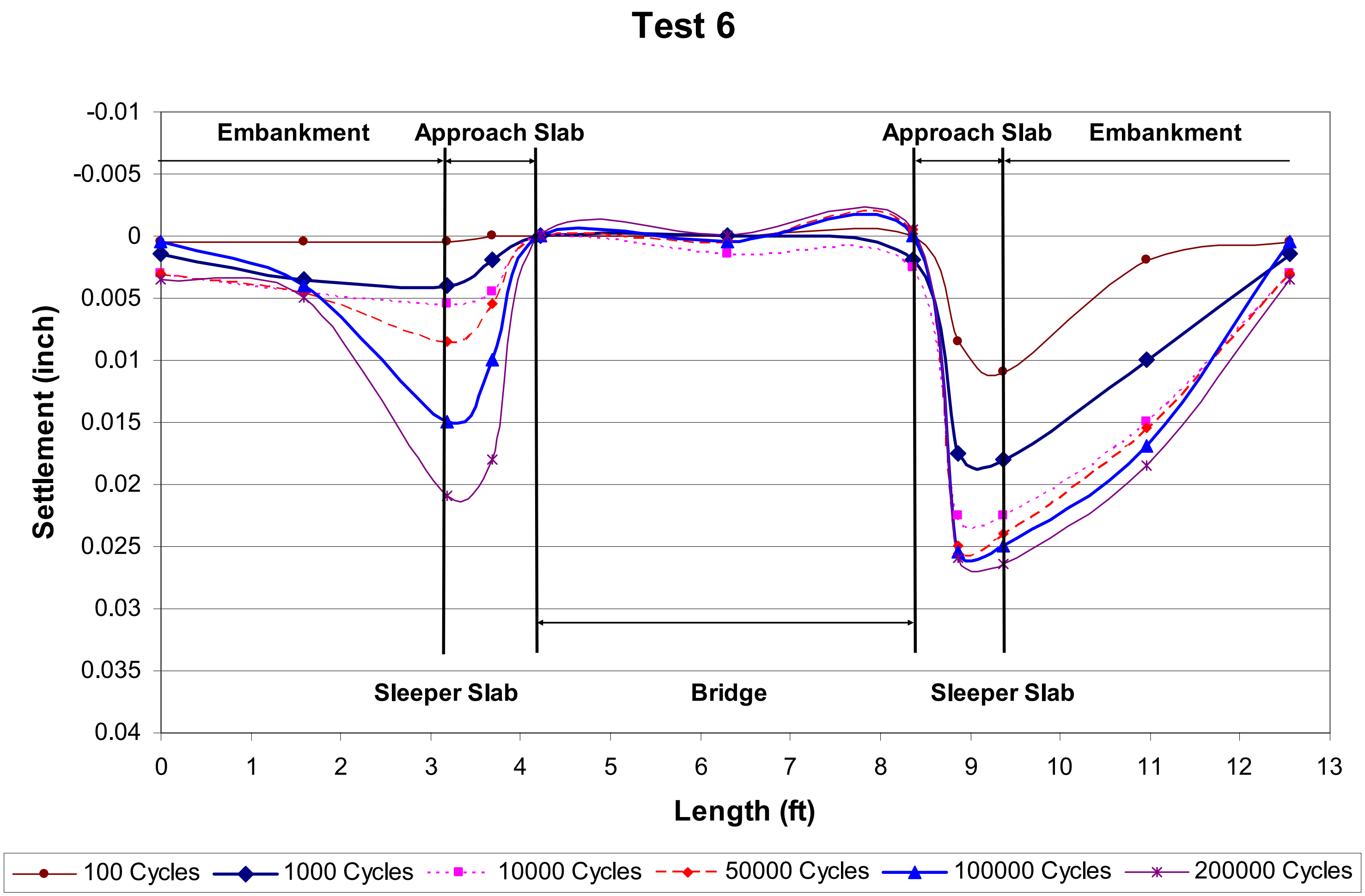


Figure 6.23. Total Profile for Test No. 6.

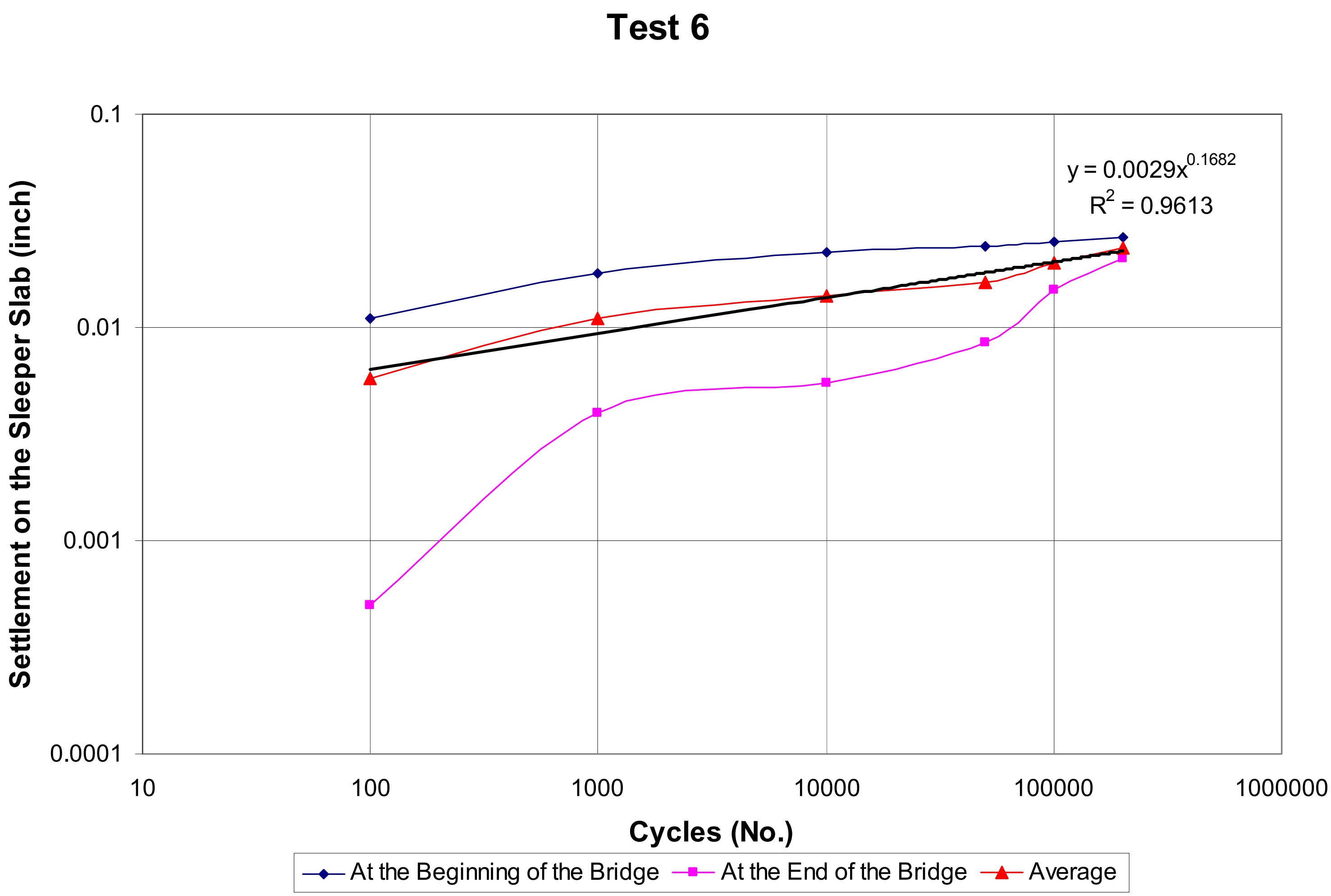


Figure 6.24. Settlement of the Sleeper Slab at Different Cycles for Test No. 6.

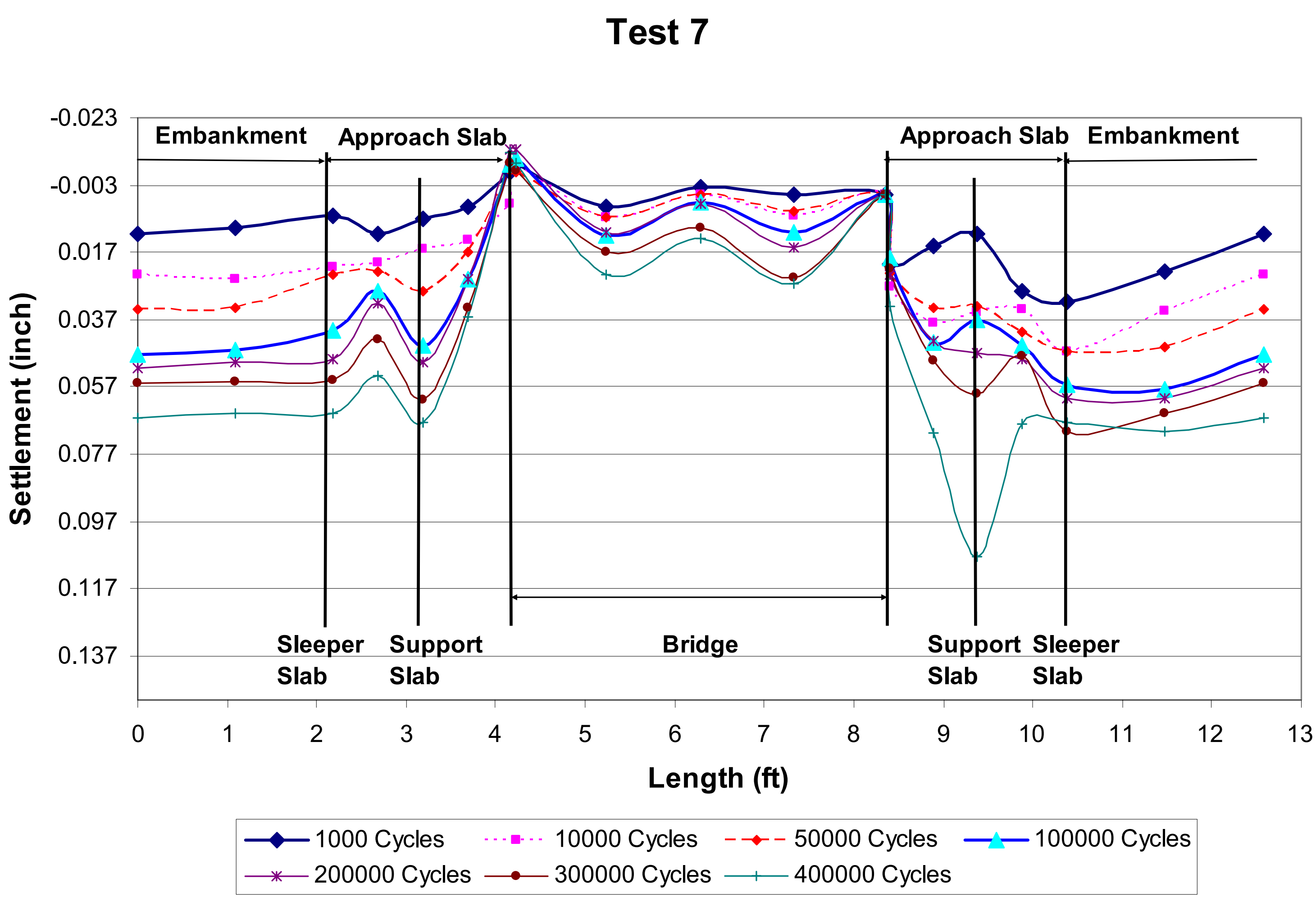


Figure 6.25. Total Profile for Test No. 7.

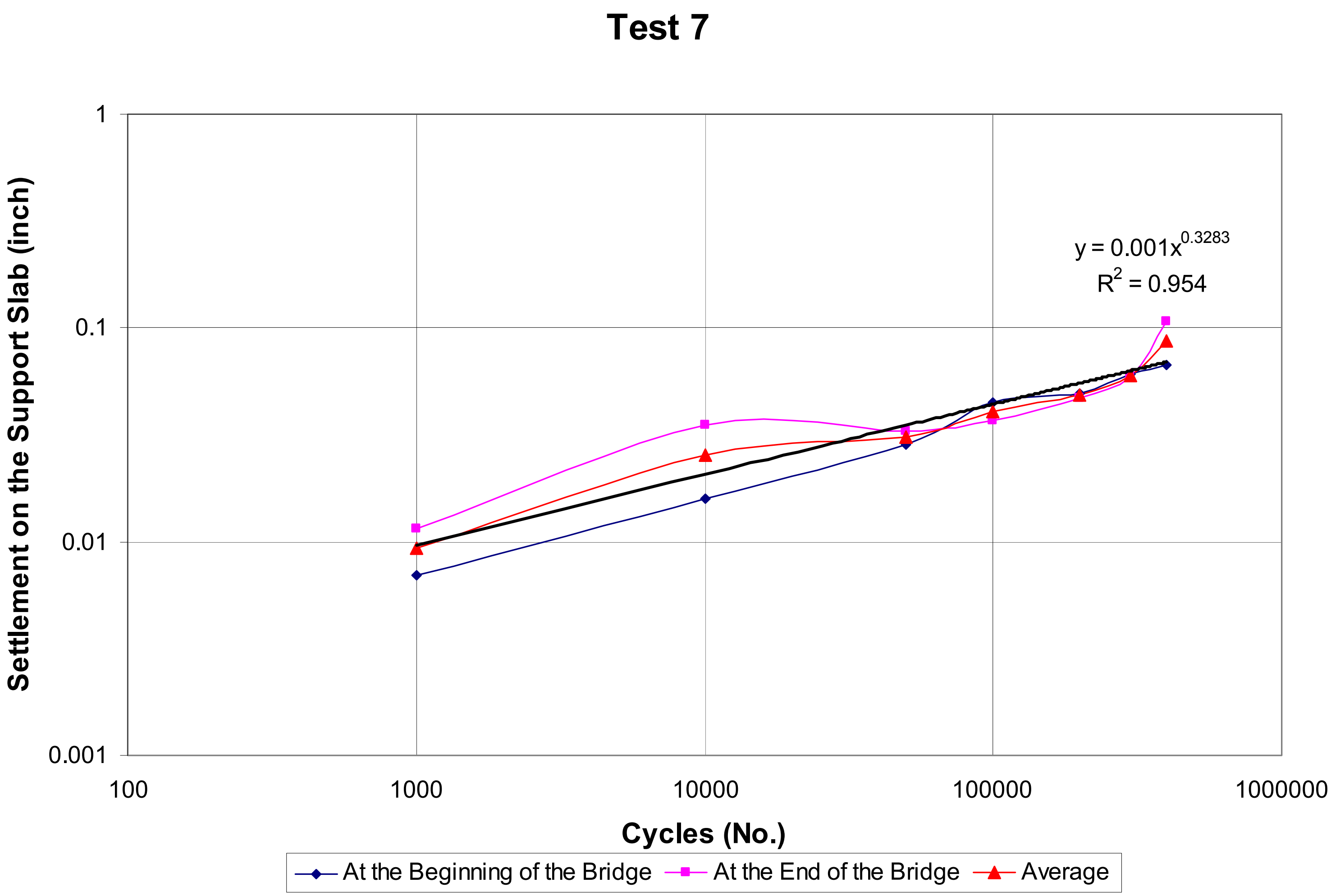


Figure 6.26. Settlement of the Sleeper Slab at Different Cycles for Test No. 7.

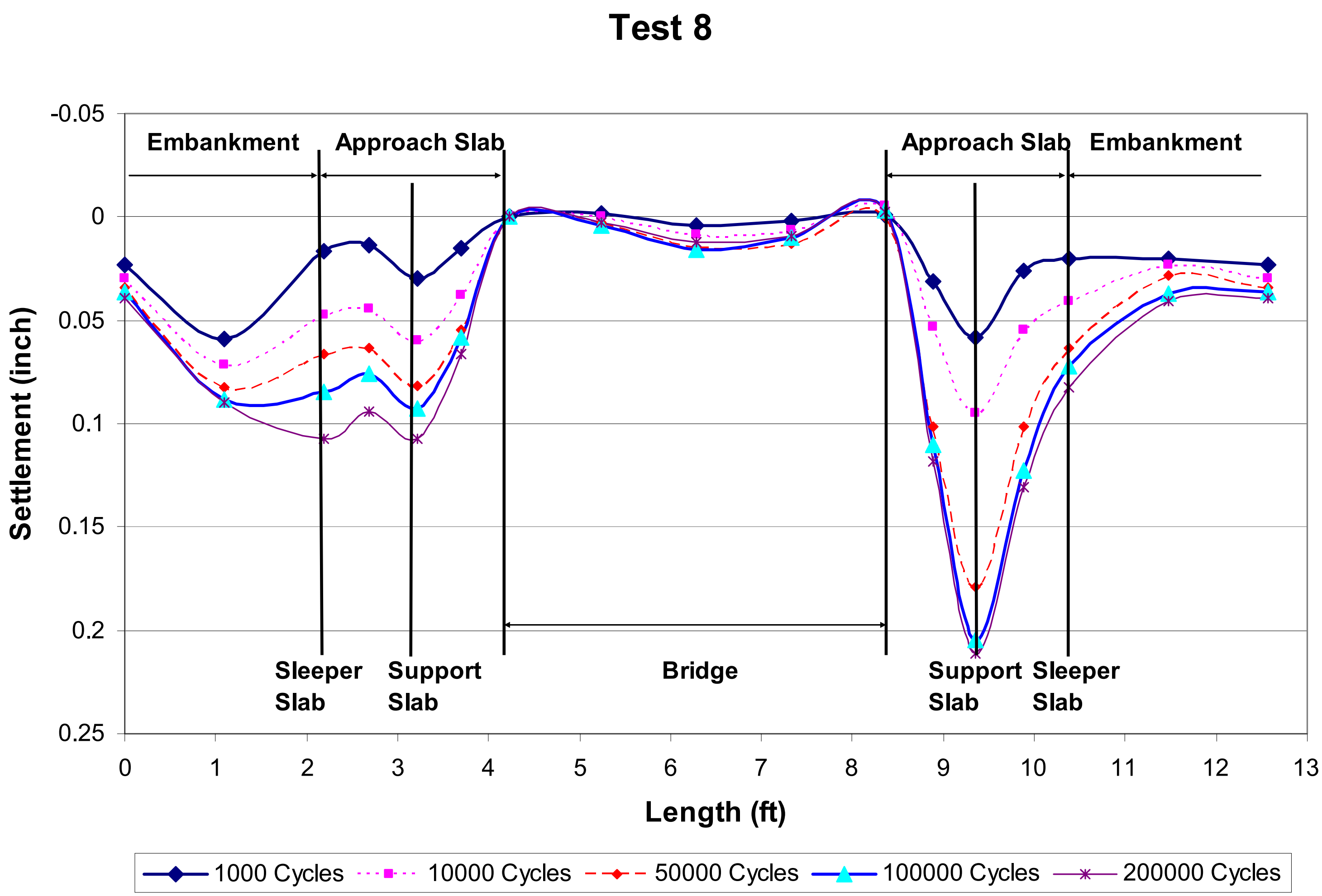


Figure 6.27. Total Profile for Test No. 8.

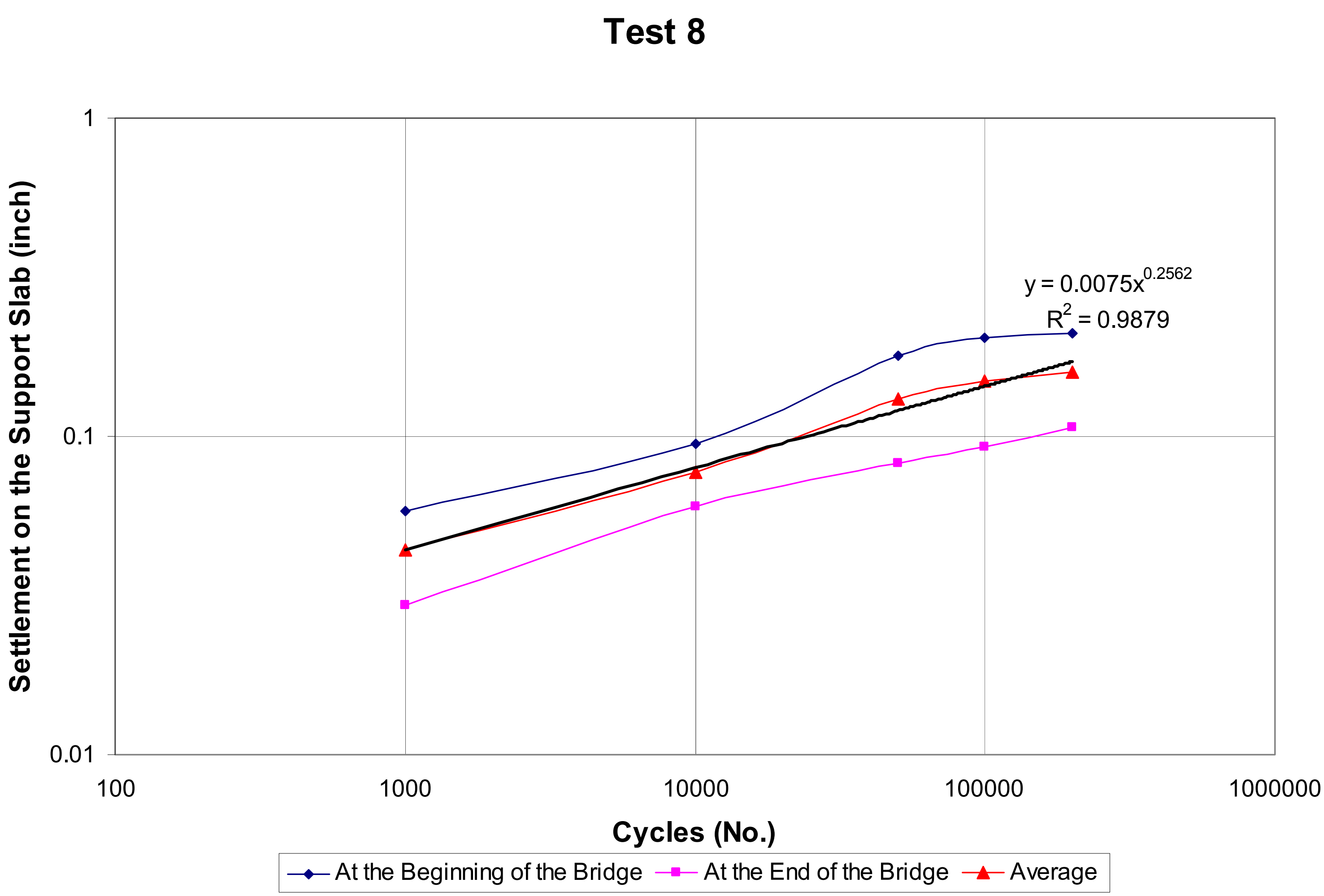


Figure 6.28. Settlement of the Sleeper Slab at Different Cycles for Test No. 8.

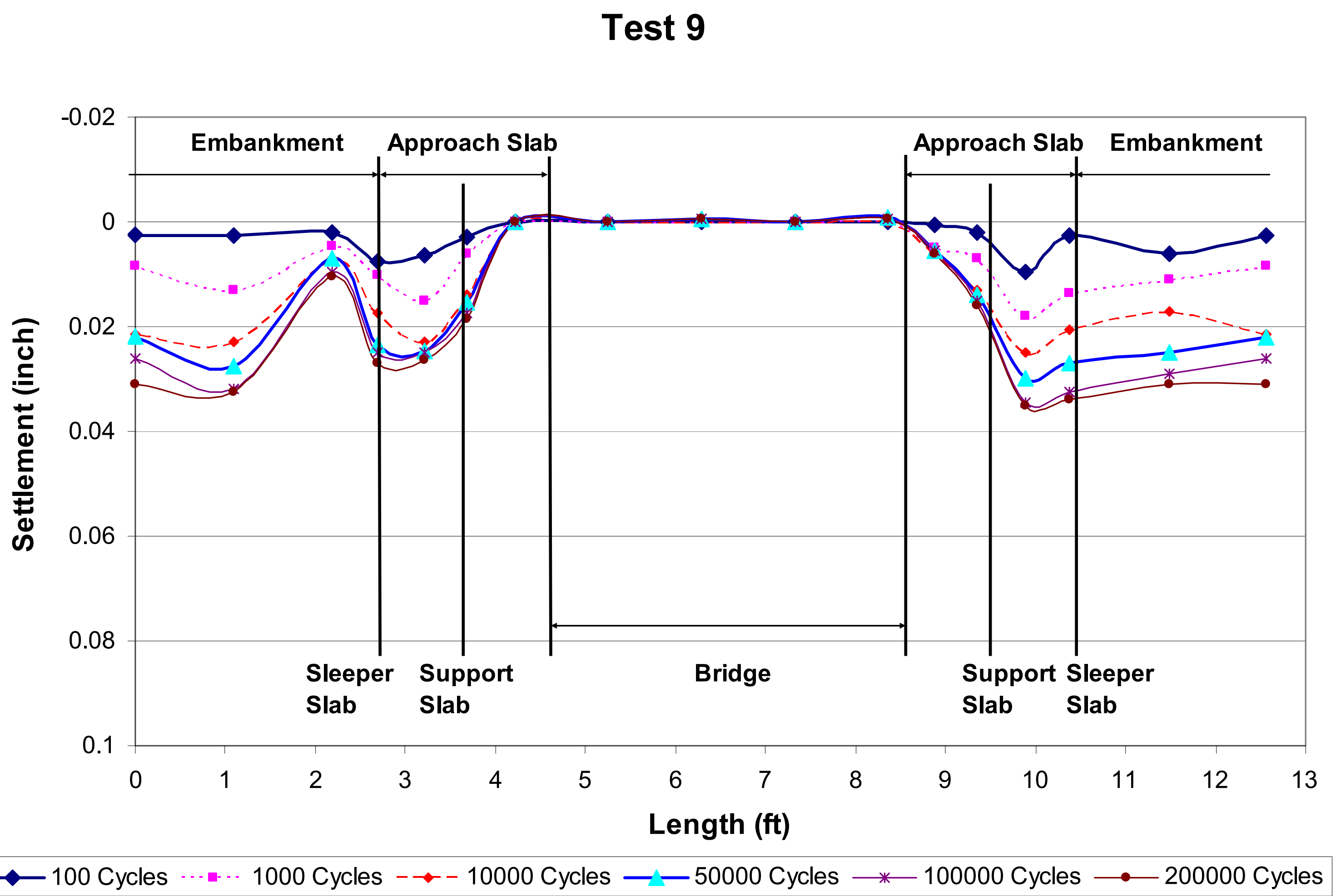


Figure 6.29. Total Profile for Test No. 9.

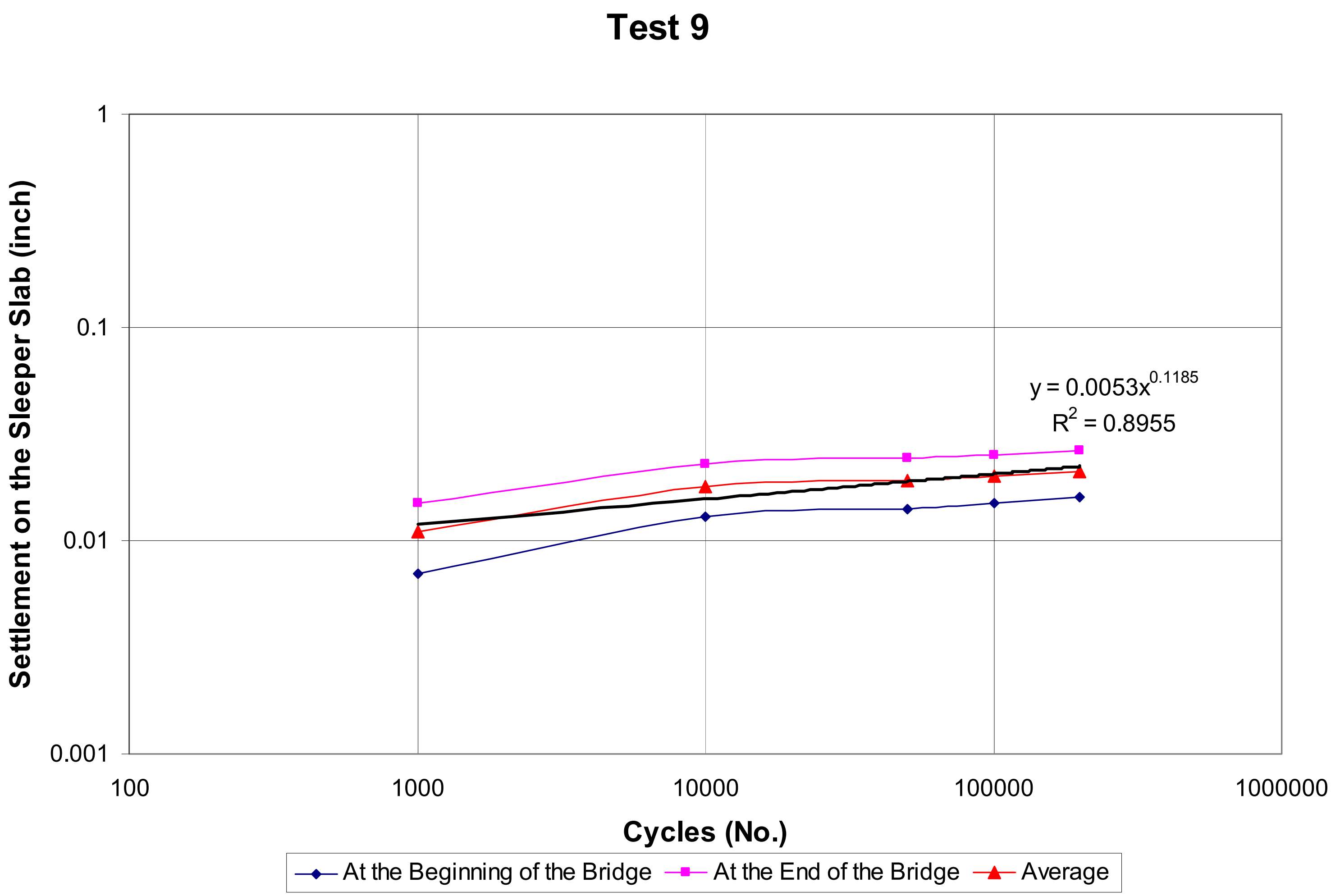


Figure 6.30. Settlement of the Sleeper Slab at Different Cycles for Test No. 9.

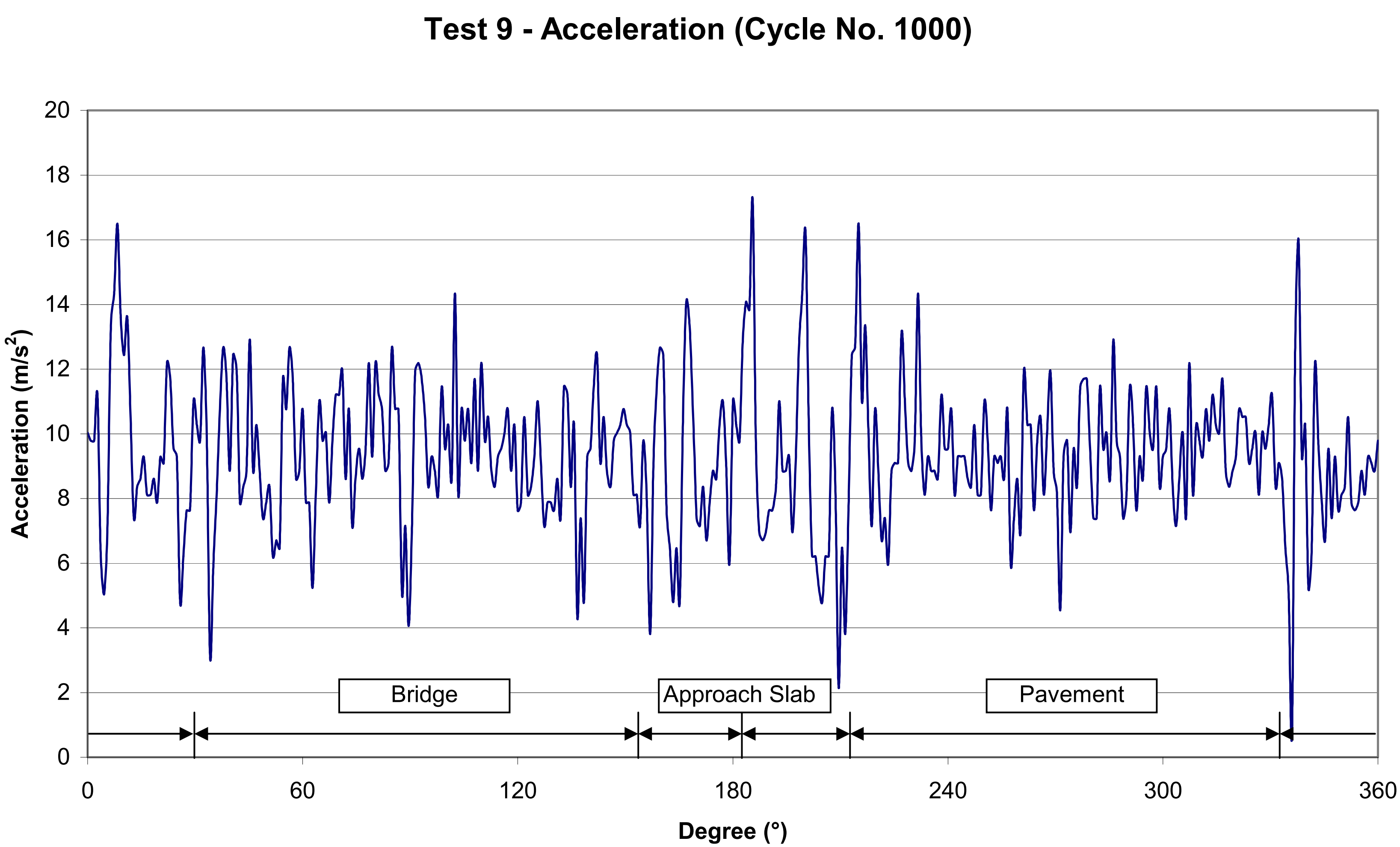


Figure 6.31. Accelerations of the Weight in Test No. 9 at Cycle No. 1,000.

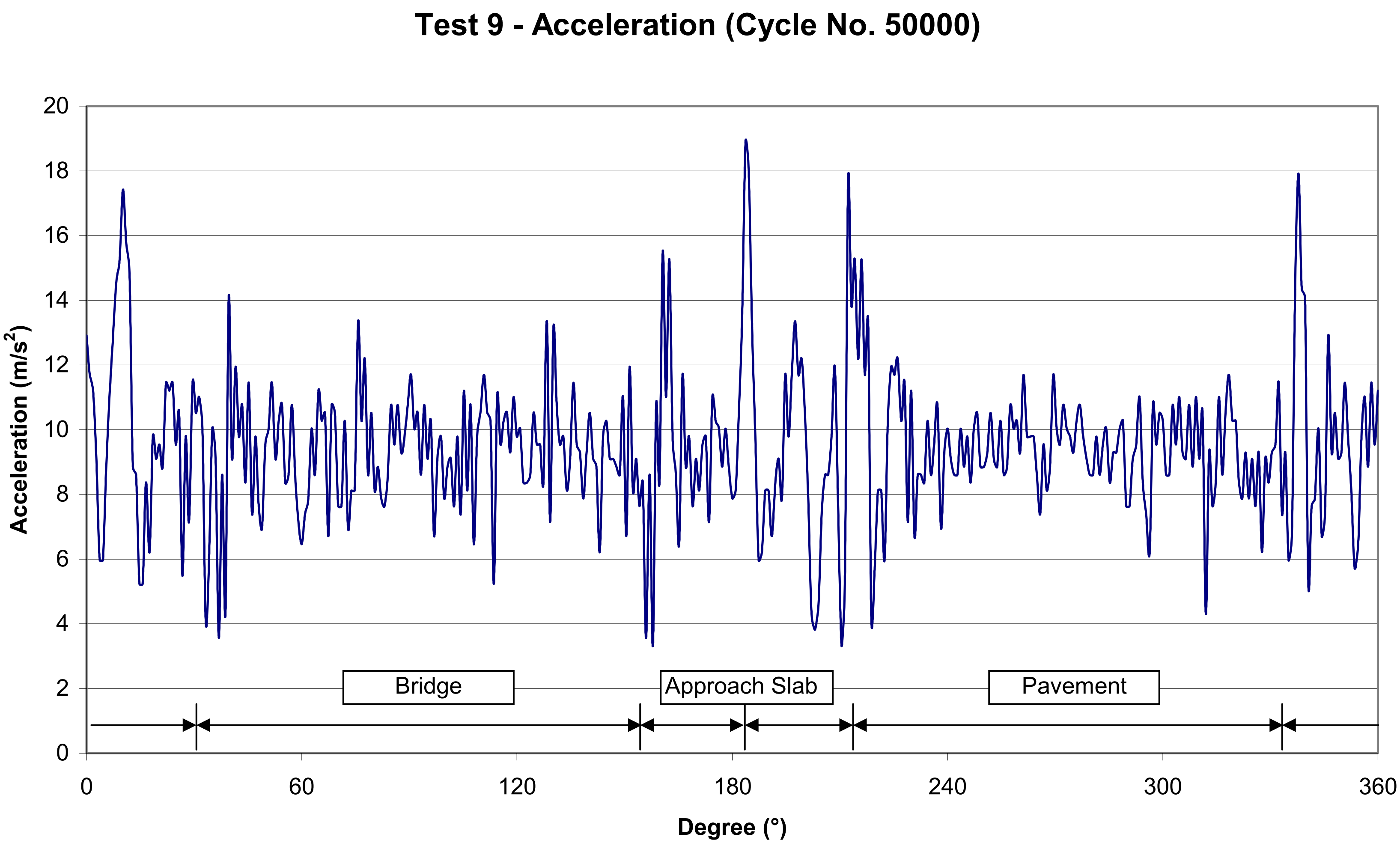


Figure 6.32. Accelerations of the Weight in Test No. 9 at Cycle No. 50,000.

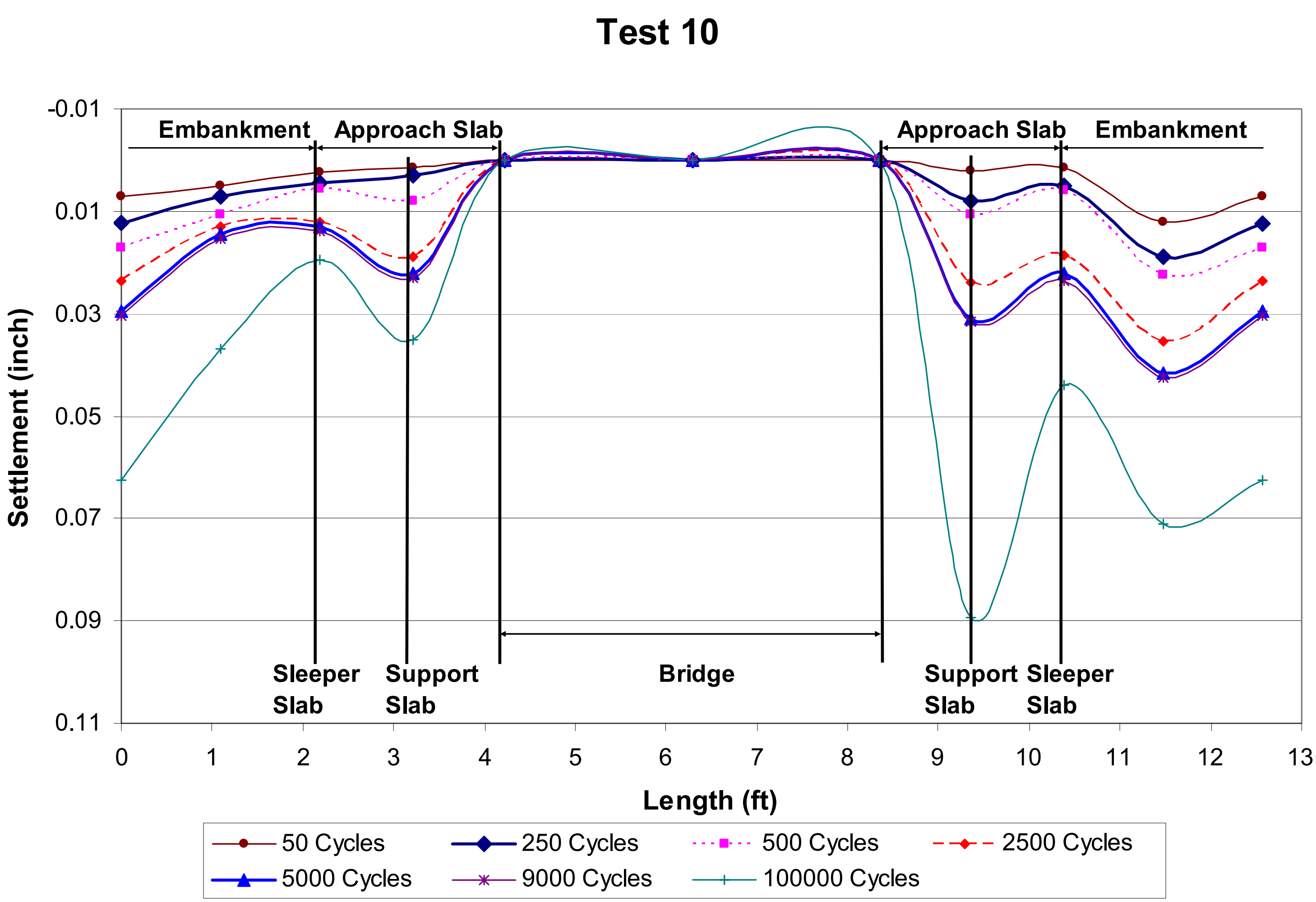


Figure 6.33. Total Profile for Test No. 10.

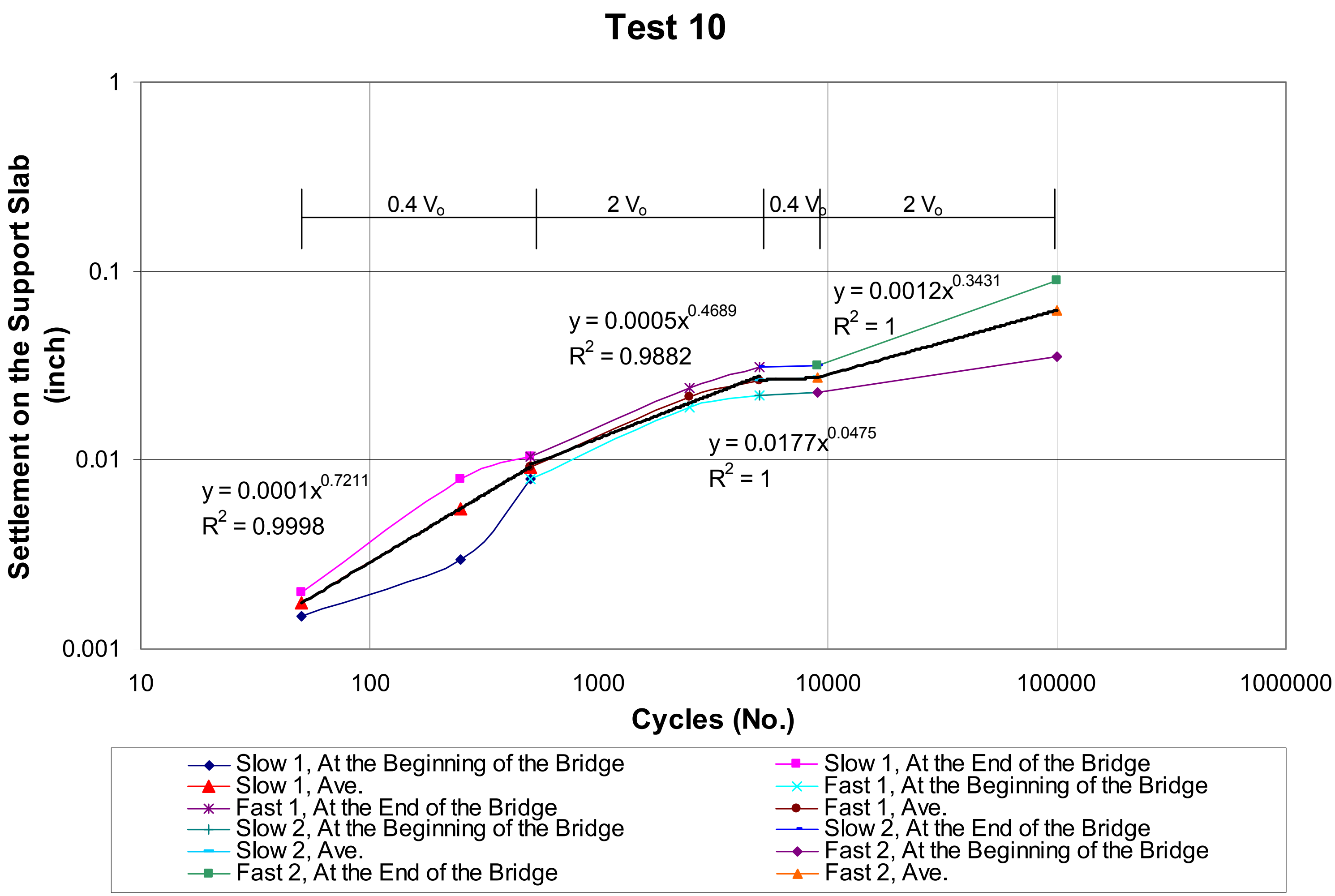


Figure 6.34. Settlement of the Sleeper Slab at Different Cycles for Test No. 10.

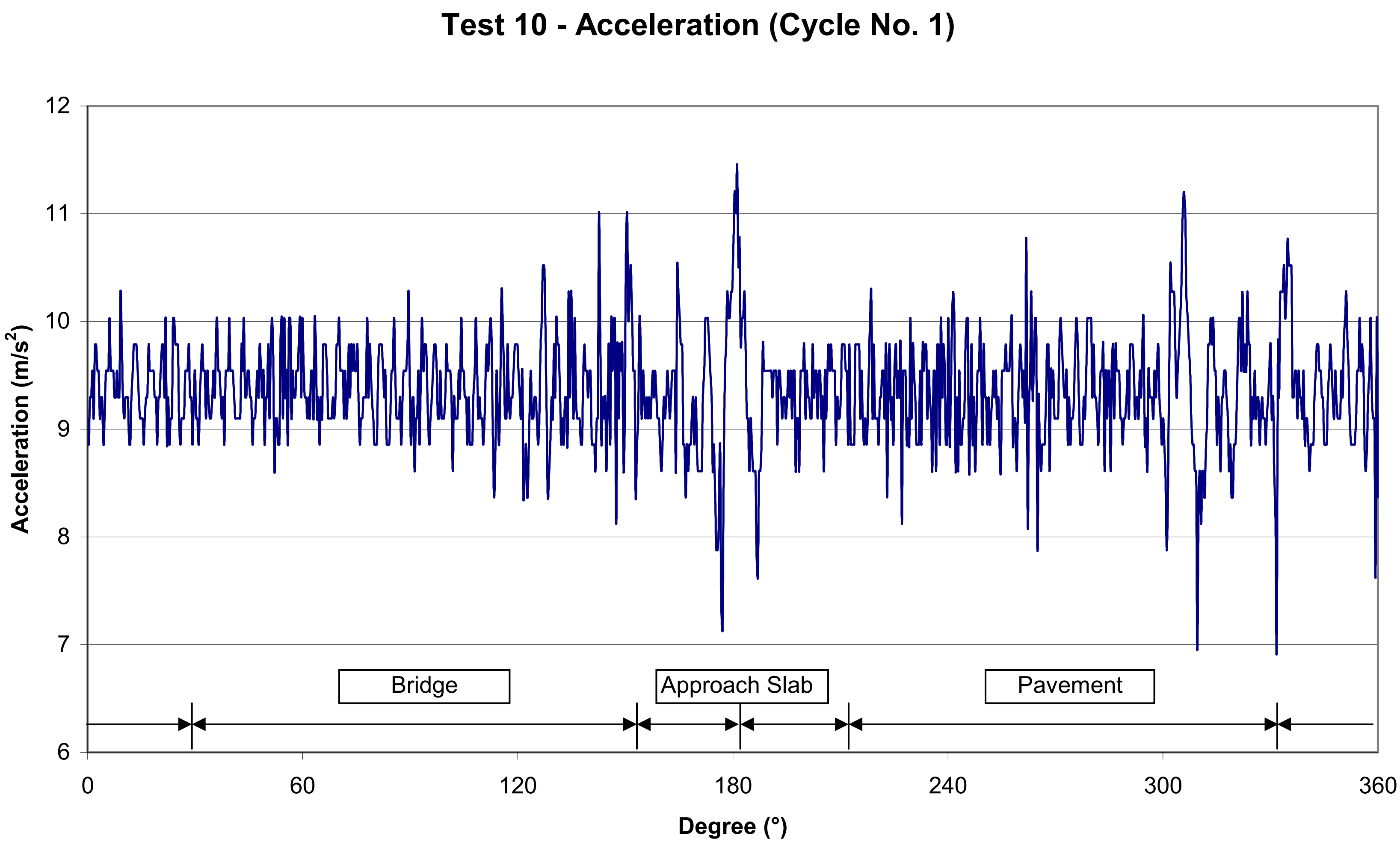


Figure 6.35. Acceleration of the Weight in Test No. 10 at Cycle No. 1.

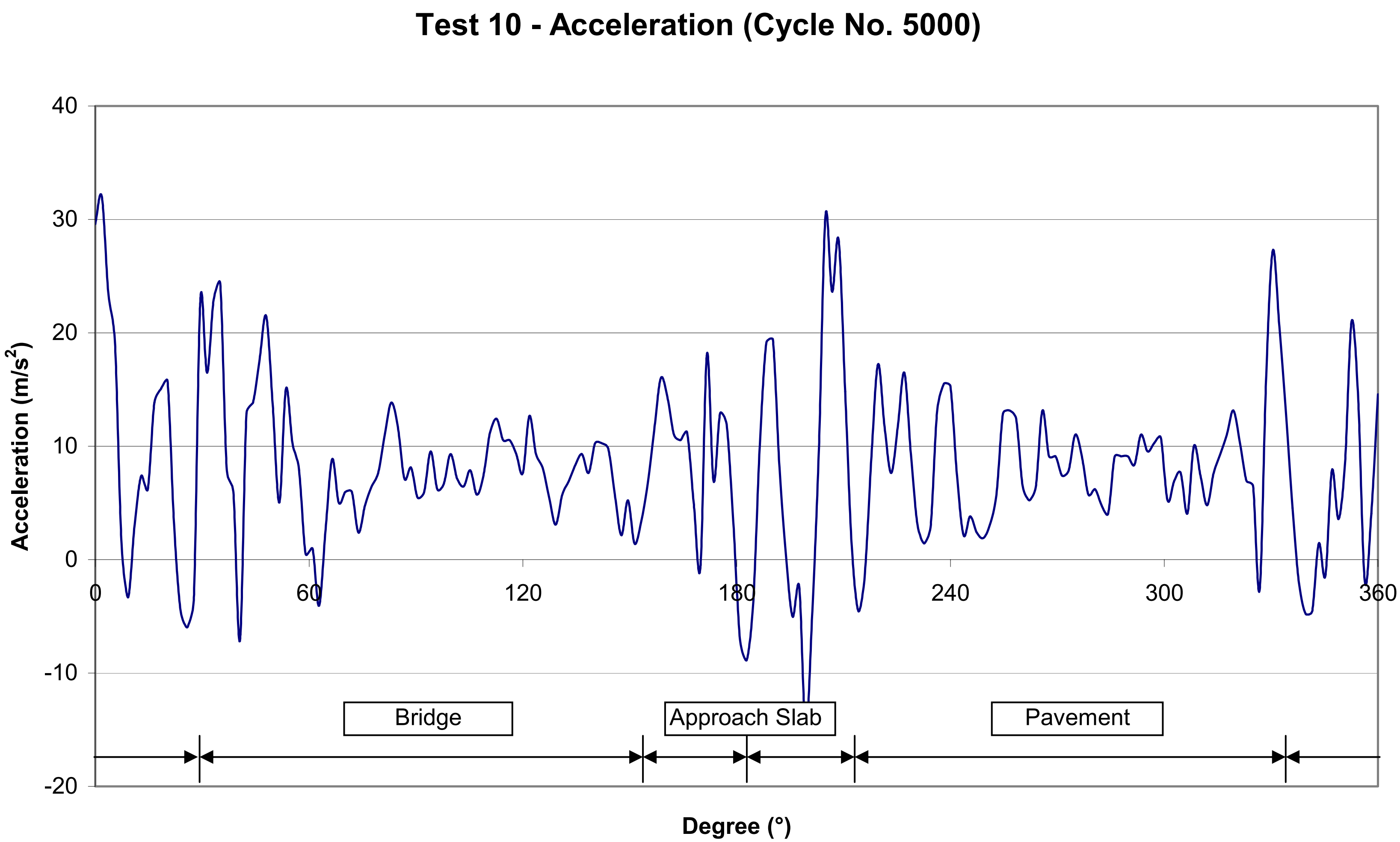


Figure 6.36. Acceleration of the Weight in Test No. 10 at Cycle No. 5,000.

Summary of Results

1. By repeating two tests with the same conditions, the researchers found that the BEST device has a good repeatability (Figure 6.37 for Tests No. 2 and No. 4).
2. The sand with the higher compaction (Test No. 1) developed less settlement at the sleeper slab than the lower compaction sand (Test No. 2) as shown in Figure 6.38.
3. The one-span approach slab (Tests No. 1, 2, and 3) with a 20-ft simulated approach slab experienced less settlement on the average than the current two-span approach slab (Tests No. 7, 8, and 9) as shown in Figures 6.39 and 6.40.
4. The velocity of the traveling wheel in the BEST device has little effect on the total settlement under the approach slab as shown in Figure 6.41 (Test No. 5 for 0.4 V_o, 2 V_o and Test No. 10 for 0.4 V_o, 2 V_o).
5. The mass loading the wheel affects the settlement as shown in Figure 6.42. (Test No. 2 with an 8 kg weight and Test No. 6 with a 1 kg weight). When the weight increased from 1 to 8 kg, the settlement at 200,000 cycles increased from 0.023 to 0.60 inch.
6. The settlement of the approach slab (the sleeper slab for the one-span approach slab and the support slab for the current approach slab) versus the number of cycles is reasonably well approximated by a straight line on a log-log plot (Figure 6.40). The slope of the line varies between 0.1377 log (settlement)/log (cycle) and 0.2957 log (settlement)/log (cycle) for these model tests.
7. The measured maximum accelerations of the BEST test were 18 m/sec^2 at V_o (1 cycle/2 seconds, 6.89 km/hr) and 32 m/sec^2 at 2 V_o (1 cycle/1 second, 13.78 km/hr). Considering that the field values of the maximum acceleration obtained by double differentiation of profilometer profile data are 30-40 m/sec^2, the measured maximum acceleration are smaller than the field acceleration. The reason is that the field acceleration represent the acceleration of the wheel below the suspension, while the model acceleration are the accelerations of the weight above the spring simulating the car suspension.

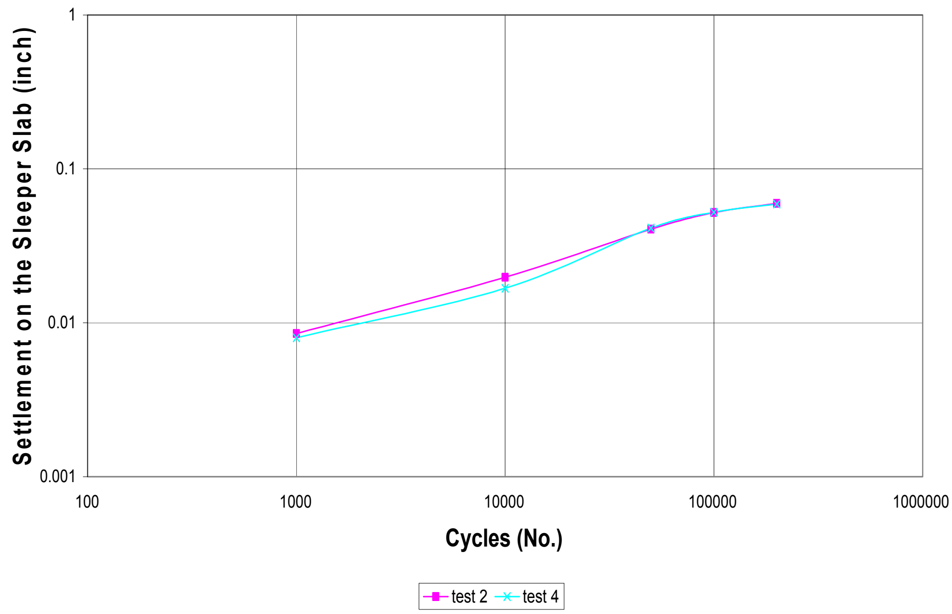


Figure 6.37. Repeatability of the BEST Test: Tests No. 2 and 4.

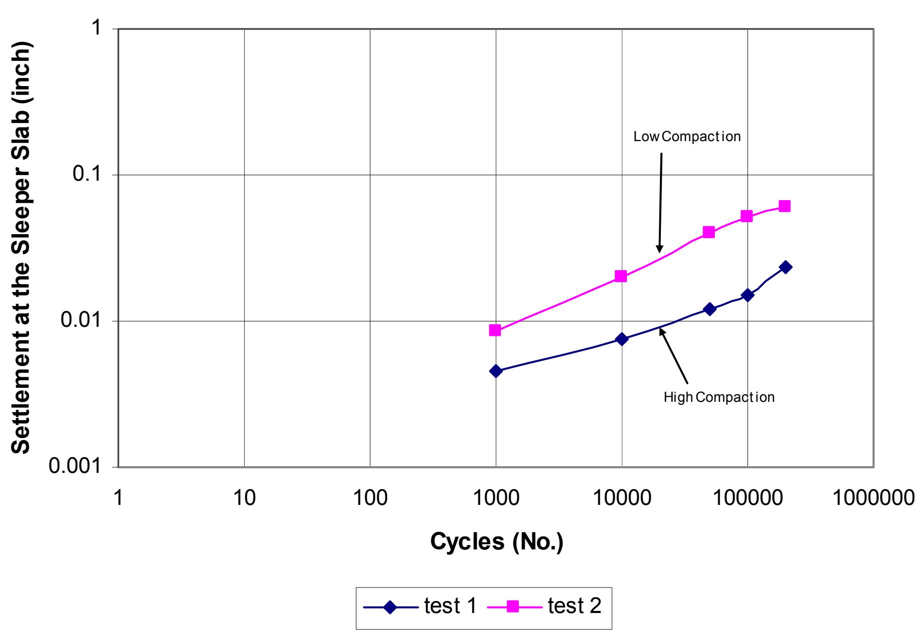


Figure 6.38. Effect of Sand Compaction: Tests No. 1 and 2.

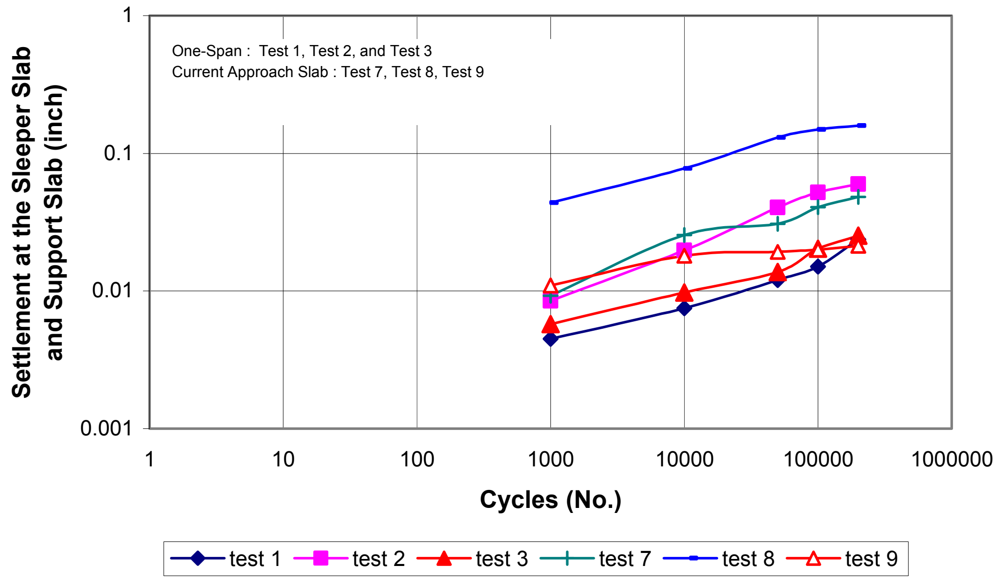


Figure 6.39. Comparison between One-Span and Two-Span Approach Slab: Test s No. 1 and 7, Tests No. 2 and 8, and Tests No. 3 and 9.

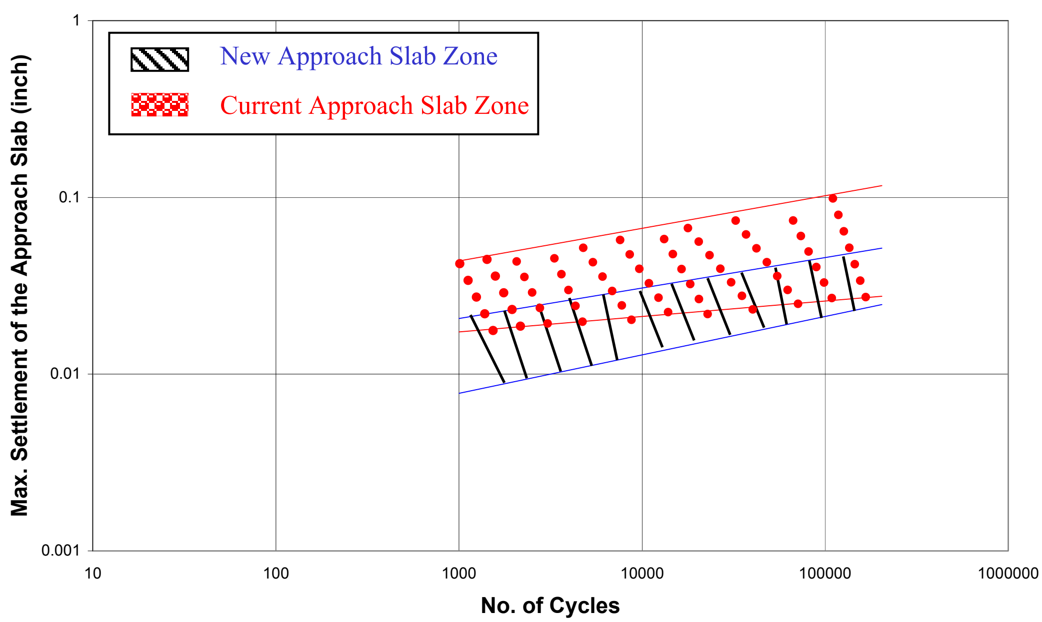


Figure 6.40. Total Results on the Sleeper Slabs and the Support Slab.

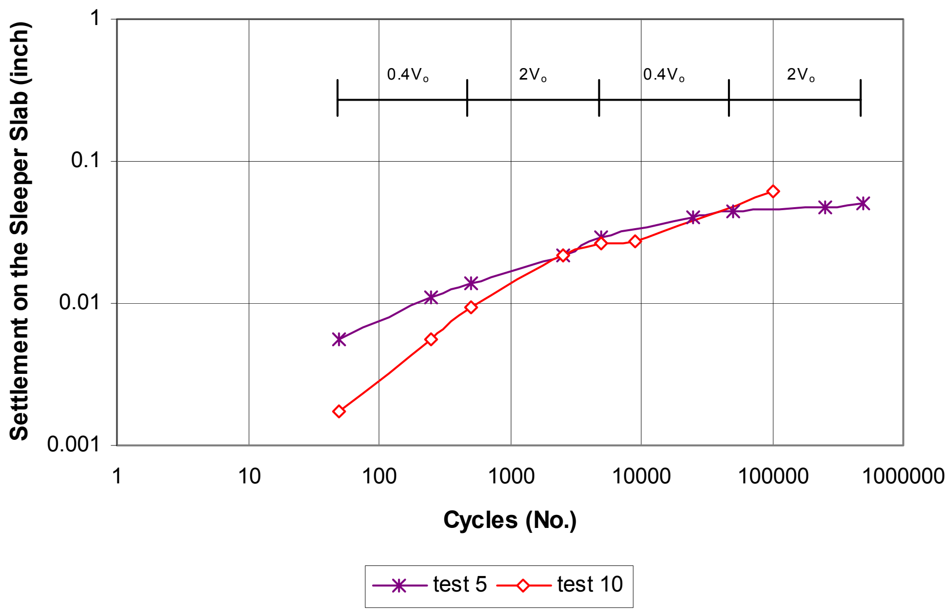


Figure 6.41. Effect of Velocity: Tests No. 5 and 10.

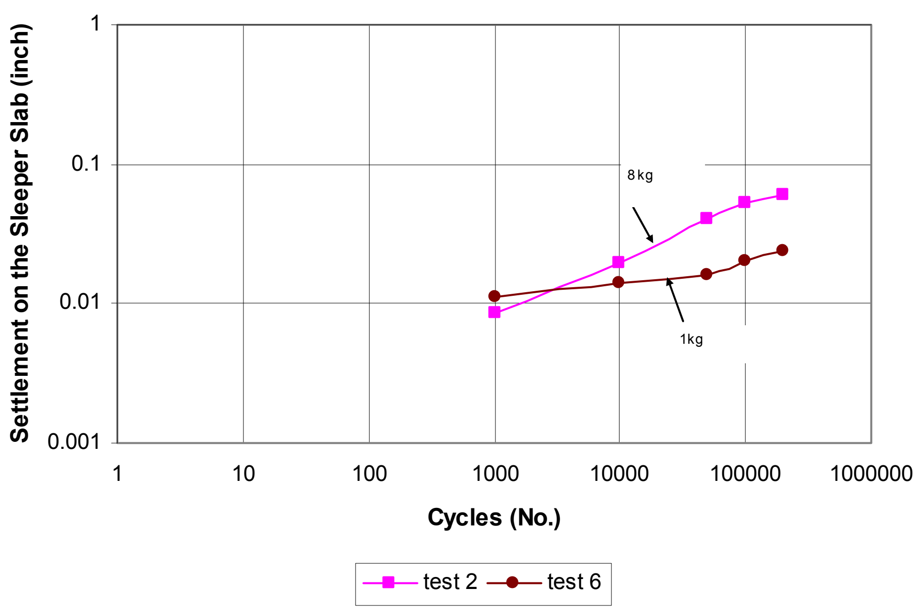


Figure 6.42. Effect of Mass: Tests No. 2 and 6.

CHAPTER 7:
CONCLUSIONS AND RECOMMENDATIONS

Conclusions from Report 4147-1

The research team investigated the bump at the end of the bridge by a literature survey, by a questionnaire distributed to the 25 districts of the Texas DOT, and by a detailed investigation of two bridge sites in Houston, Texas.

The literatures surveyed led to the following conclusions:

1. On the average, 25 percent of all bridges in the USA are affected by the bump problem.
2. The maintenance cost for the bump problem in the USA is estimated at 100 million dollars per year (1997 dollars).
3. The main reasons for the development of a bump are the settlement of the embankment due to a weak natural soil or to the compression of the embankment fill, voids under the pavement due to erosion, and abutment displacement due to pavement growth, slope instability, or temperature cycles.
4. The bump is more severe if there is a high embankment, an abutment on piles, high average daily traffic, soft natural soil, intense rain storms, extreme temperature cycles, and steep approach gradients.
5. The bump is less severe when there is an approach slab, appropriate fill material, good compaction or stabilization, effective drainage, good construction practice and inspection, and an adequate waiting period between fill placement and paving.
6. A tolerable bump has a slope of 1/200 or less.

The best approach recommended in the literature is:

1. Treat the bump problem as a stand-alone design issue and make prevention a design goal.
2. Assign the responsibility of this design issue to an engineer.
3. Stress teamwork and open-mindedness among the geotechnical, structural, pavements, construction, and maintenance engineers.

4. Carry out proper settlement versus time calculations.
5. If differential settlement is excessive, design an approach slab.
6. Provide for expansion/contraction between the structure and the approach roadway (fabric reinforcement, flow fill).
7. Design a proper drainage and erosion protection system.
8. Use and enforce proper specifications.
9. Choose knowledgeable inspectors especially for geotechnical aspects.
10. Perform a joint inspection including joints, grade specifications, and drainage.

The questionnaire results led to the following conclusions:

1. On the average, 24.5 percent of the bridges in Texas have a bump problem.
2. The maintenance cost for the bump problem in Texas is estimated at 6.3 million dollars per year (2001 dollars).
3. The number one reason for the bump is the settlement of the embankment fill, followed by the loss of fill by erosion.
4. The problem is worse when the embankment is high and the fill is clay.
5. The problem is minimized when an approach slab is used and the fill behind the abutment is cement stabilized.

Two bridge overpass sites on major highways in Houston were subjected to a detailed investigation. Both bridge sites had articulated two-span approach slabs with a wide flange beam. The investigation led to the following conclusions:

1. The profilometer gave bump amplitudes varying from 1.15 to 2.35 inches on April 2001 and from 0.76 to 2.12 on March 2002, transition slopes as steep as 1/100; international roughness indices as high as 820, indicating a rough unpaved road condition; and present serviceability indices of 0.00, indicating really poor condition.
2. The profilometer test performed one year after the first one indicated that some of the bumps had decreased and some had stayed the same, while others had increased. Therefore, bumps are dynamic features that may be tied to the

weather through the shrink-swell nature of some soils used for embankment fills.

3. Close to the bridge abutment, the cone penetrometer resistance was 33.8 percent lower on the average and the water content was 10.5 percent higher on the average than the values away from the abutment.
4. The compaction level within the embankment below the bump averaged 96 percent of the Standard Proctor maximum dry unit weight.
5. The soil of the embankment fill had a PI varying from 8.52 to 33.77 with an average equal to 20.96.
6. The ground penetrating radar indicated that there were no voids under the pavement.

The data seem to indicate that the soil near the abutment is more exposed to water than the soil away from the abutment. This exposure leads to a higher water content, a lower strength, and therefore, a higher compressibility of the soil, which leads to the bump.

A bump rating (BR) number and a bump index (BI) were developed as part of this research project. The bump number goes from 0 for no bump to 4 for a dangerous bump and is typically obtained by guessing at the BR number after riding over the pavement at full speed. The number refers to the differential settlement in inches between the low and high point of the bump. The bump ratings at the two sites investigated ranged from 0 to 2. The BI gives an estimate for the likelihood that a bump will develop for a given situation. The equation giving the bump index includes the height of embankment, average daily traffic (ADT), bridge life, average yearly precipitation, temperature cycle, resistance of abutment, resistance of embankment, and gradient of approach. Further research is needed if the coefficients involved in that equation are to be determined.

Conclusions from This Report (4147-2)

Researchers summarized the first year work by reviewing the literature survey, the questionnaire results, and the investigation of two bridge sites in Houston with significant bump problems. It led to the following conclusions:

1. Twenty five percent of all bridges in the USA and in Texas are affected by the bump problem.
2. The maintenance cost for the bump problem in the USA is estimated at 100 M$ per year (1997) and 7.0 M$ per year in Texas (2001).
3. A tolerable bump has a slope of 1/200 or less.
4. In Texas the number one reason for the bump is the settlement of the embankment fill followed by the loss of fill by erosion.
5. The problem is worse when the embankment is high and the fill is clay.
6. The problem is minimized when an approach slab is used and the fill behind the abutment is cement stabilized.
7. The soil near the abutment of the two sites studied was weaker and wetter than the soil away from the abutment.
8. The soil near the abutment of the two sites studied had a relatively high PI for an embankment fill.
9. There were no voids under the pavement according to the ground penetrating radar.
10. The vertical acceleration of the wheel of the vehicle reached 4 g's at one of the bump sites.
11. A bump rating number and a bump index number are proposed to document the severity of existing bumps and to evaluate the likelihood of developing a bump at a site.

Researchers surveyed planning, design, and construction, maintenance and rehabilitation practices for the approach slab. It led to the following conclusions:

1. For embankments higher than 15 ft, the recommended boring spacing is a maximum of 200 ft. For each bridge abutment, a maximum of two borings is

recommended, and additional borings are suggested when the abutment exceeds 100 ft in length or has wingwalls more than 20 ft long.

2. Two major design concepts, conventional bridges and integral abutment bridges, are currently used for road bridges. The conventional design type has a superstructure resting on an abutment at each end, but the integral abutment bridges are connected with superstructure and abutment.
3. Some states specify fill with a maximum PI of 15 and fewer than 40 percent fines within 150 ft of an abutment wall, and the required relative compaction is increased to 95 percent from 90 percent within approach embankments.
4. Five types of abutment are in use: closed or high abutment, stub or perched abutment, pedestal or spill-through abutment, integral abutment, and mechanically stabilized abutment.
5. Approach slabs are used in about 80 percent of new bridges (Schaefer and Koch, 1992). Most approach slabs are 20 to 40 ft long and 9 to 12 ft thick.
6. The approach embankment can be constructed either before or after the bridge and the abutment. Closed, spill-through, and integral abutments require that the abutment be built first, but perched and MSE abutments are constructed after the embankment is finished.
7. Moulton et al. (1985) suggest a tolerable angular distortion of 1/250 for continuous-span bridges and 1/200 for simply supported spans.
8. Most bridges designed in Texas have stub or perched abutments with the approach slab and wide flange terminal joint.

Researchers proposed a new approach slab that has a one-span slab. They arrived at this new design by reviewing the components related to the settlement at the bridge approach slab expansion joint, performing numerical analyses, and conducting model scale simulations.

The numerical analyses led to the following conclusions:

1. The presence of the abutment wall on piles creates a major difference in settlement between the abutment wall and the embankment.

2. The differential settlement is drastically reduced in the absence of the wall.
3. The transition zone is about 40 ft with 80 percent of the maximum settlement occurring in the first 20 ft for a uniform load case.
4. The size of the sleeper slab and support slab influences the settlement of the slab when load is applied to the slab. The optimum width of both slabs is 5 ft. The height of the embankment influences the settlement of the embankment.

The new proposed approach slab has the following characteristics:

1. The new approach slab is 20 ft long and has one span from the abutment to the sleeper slab.
2. It is designed to carry the full traffic load without support on the soil except at both ends; the support slab is removed and the wide flange is kept on the embankment side as a temperature elongation joint.
3. This new approach slab will simplify construction, will be less expensive, and will place less emphasis on the need for very good compaction close to the abutment wall.

The Bridge to Embankment Simulator of Transition device, which is a 1/20th scale model of the typical transition, was designed, built, and used to simulate the problem. The results of the BEST tests led to the following conclusions:

1. The proposed new approach slab (one-span) with a 20 ft simulated approach slab gave a smaller bump than the current two-slab approach slab.
2. The soil with the higher compaction developed less bump at the sleeper slab than the lower compaction soil.
3. The bump size increases with the number of cycles in a straight line on a log-log plot.
4. The maximum acceleration the BEST test recorded, 32 m/sec^2 at the velocity of 13.78 km/hr, was smaller than the maximum field acceleration, 40 m/sec^2, obtained by double differentiation of the profilometer data.

Final Recommendations

The following recommendations are made for the zone located within 100 ft from the abutment:

1. Use quality backfill: PI less than 15, less than 20 percent passing sieve #200, coefficient of uniformity larger than 3.
2. Compact the soil to 95 percent of Modified Proctor, controlled by inspection, with an increased rate of measurement more frequent than required by current TxDOT specifications.

If such a quality backfill cannot be achieved, the embankment fill within that 100-ft zone should be cement stabilized.

The following recommendation is made for the approach slab.

1. Use a single-slab approach slab that is at least 20 ft long and 13 inches thick. The articulation that exists in the current approach slab is removed, and the wide flange is kept on the embankment side as a temperature elongation joint. Design the approach slab to handle the full load in free span.

REFERENCES

AASHTO (1984), *Manual on Subsurface Investigation* (Draft), American Association of State Highway and Transportation Officials, Washington, D.C.

"A Look at Our Nation's Bridge," Transportation Builder (February 1995), p. 29.

Barenberg, E. J. and B. P. Hazarida (1976), "Use of University of Illinois Test Track to Evaluate Pavement Performance," Transportation Research Rec. 574, TRB, Washington, D.C., pp. 38-50.

Bellin, J. (1993), "Bridge Deck Joints," Oregon Department of Transportation, Bridge Section, Salem, p. 9.

Briaud, J-L., R. W. James, and S. B. Hoffman (1997), "NCHRP Synthesis 234: Settlement of Bridge Approaches (The Bump at the End of the Bridge)," Transportation Research Board, National Research Council, Washington, D.C., p. 10.

Briaud, J-L., and Y. J. Lim (1997), "Soil-Nailed Wall under Piled Bridge Abutment: Simulation and Guidelines," J. Geotech. Engrg. Div., ASCE, 123(11), pp. 1043-1050.

Brown, S. F. and B. V. Brodrick (1981), "Nottingham Pavement Test Facility," Transportation Research Rec. 574, TRB, Washington, D.C., pp. 67-72.

Burke, M. P. (1987), "Bridge Approach Pavements, Integral Bridges and Cycle Control Joints," Presented at the 66th Annual Meeting of the Transportation Research Board, Washington, D.C.

Chini, S. A., A. M. Wolde-Tinsae, and M. S. Aggour (1993), "Drainage and Backfill Provisions for Approachs to Bridges," Transportation Research Board, National Research Council, Washington, D.C., pp. 45-53.

CRSI (2000), "Summary of CRCP Design and Construction Practices in the U.S.," Research Series No. 8, Schaumburg, Illinois from the World Wide Web: http://www.crsi.org/PDF/research_series_8.pdf/.

Elias, V. and B. R. Christopher (1996), "Mechanically Stabilized Earth Walls and Reinforced Earth Slopes," Report FHWA-DP.82-1, Federal Highway Administration, Washington, D.C.

Hopkins, T. C. (1985), "Long-Term Movements of Highway Bridge Approach Embankments and Pavements" Research Report UKTRP-85-12, Kentucky Transportation Research Program, p. 149.

Hoppe, Edward J. (November 1999), "Guidelines for the Use, Design, and Construction of Bridge Approach Slabs," VTRC Report No. 00-R4, Virginia Transportation Research Council, Charlottesville, Virg., p. iii. Retrieved June 15, 2002, from the World Wide Web: http://www.virginiadot.org/vtrc/main/online_reports/pdf/00-r4.pdf.

Horvath, S. John (2000), "Integral-Abutment Bridges: Problems and Innovative Solutions Using EPS Geofoam and Other Geosynthetics," Manhattan College Research Report No. CE/GE-00-2, Bronx, New York, p. 10.

Kramer, S. L. and P. Sajer (December 1991), "Bridge Approach Slab Effectiveness," Report No. WA-RD 227.1, Washington State Department of Transportation, Olympia, Washington, p. 223.

Moulton, L. K., H. V. S. Gangarao, and G. T. Halvorsen (1985), "Tolerable Movement Criteria for Highway Bridges," Report No. FHWA/RD-85/107, FHWA, Washington, D.C., p. 109.

Paterson, W. D. O. (1972), "Deformations in Asphalt Concrete Wearing Courses Caused by Traffic," Proc., 3rd Int. Conf. on Struct, Design of Asphalts Pavement, London, England, pp. 317-325.

Road Transport Research (1985), "Full Scale Pavement Tests," Rep. Organization for Economic Co-Operation and Development (OECD), Paris, France.

Schaefer, V. R. and J. C. Koch (November 1992), "Void Development under Bridge Approaches," Report No. SD90-03, South Dakota Department of Transportation, Pierre, p. 147.

Shackel, B., and M. G. Arora (1978), "The Application of a Full-scale Road Simulator to the Study of Higyway Pavements," Australian Road Res., Vermont South, Victoria, Australia, 8(2) pp. 17-31.

Stark, T. D., S. M. Olsen, and J. H. Long (April 1995), "Differential Movement at the Embankment/Structure Interface-Mitigation and Rehabilitation," Report No. IAB-H1, FY 93, Illinois DOT, Springfield, Illinois, p. 297.

Tadros, M. K. and J. V. Benak (December 1989), "Bridge Abutment and Approach Slab Settlement (Phase 1)," Nebraska Department of Roads, Lincoln, Nebraska, p. 89.

TRB (1969), "NCHRP Synthesis of Highway Practice 2: Bridge Approach Design and Construction Practices," Transportation Research Board, National Research Council, Washington, D.C., p. 30.

TRB (1971), "NCHRP Synthesis of Highway Practice 8: Construction of Embankments," Transportation Research Board, National Research Council, Washington, D.C., p. 38.

TxDOT (October 2000), "TxDOT Manual System-Geotechnical Manual," Texas Department of Transportation, Austin, Texas. Retrieved June 15, 2002, from the World Wide Web: http://manuals.dot.state.tx.us/dynaweb/colbridg/geo/@Generic__BookView.

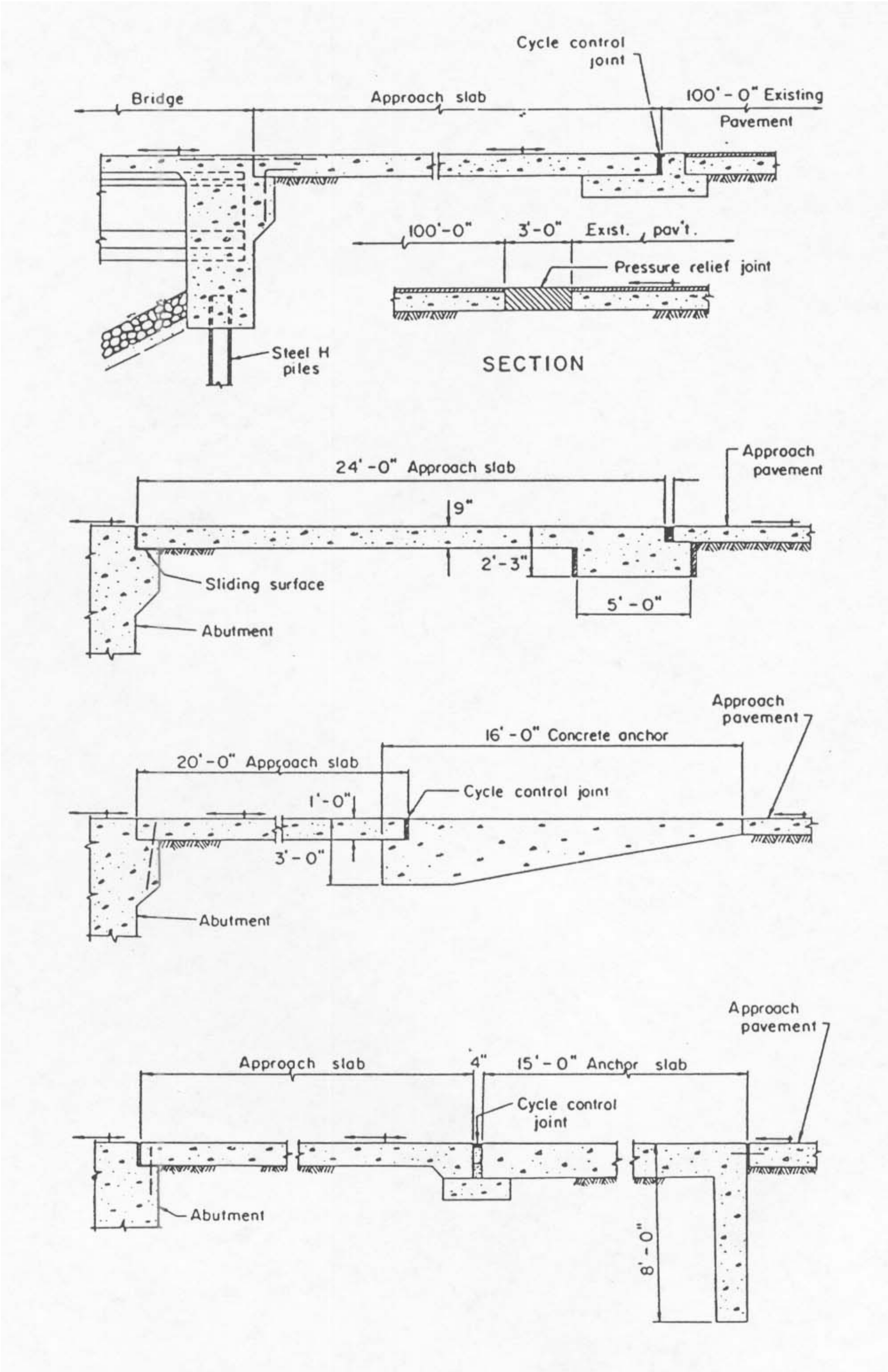
TxDOT (October 2000), "TxDOT Manual System-Bridge Design Manual," Texas Department of Transportation, Austin, Texas. Retrieved June 15, 2002, from the World Wide Web: http://manuals.dot.state.tx.us/dynaweb/colbridg/des/@Generic__BookView

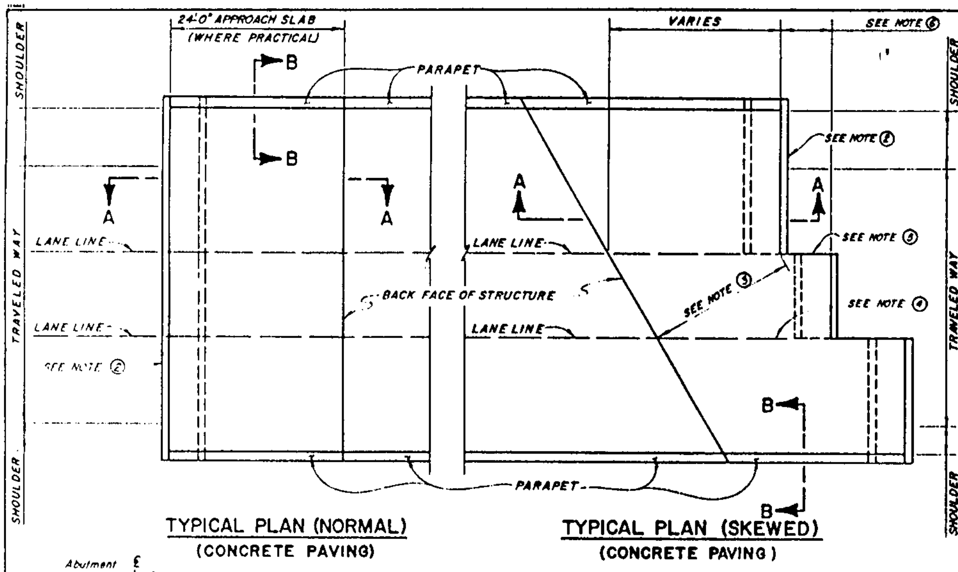
Wahls, H. E. (July 1990), "NCHRP Synthesis of Highway Practice 159: Design and Construction of Bridge Approachs," Transportation Research Board, National Research Council, Washington, D.C.

APPENDIX A	**EXAMPLE OF APPROACH SLAB DETAILS**
APPENDIX B	**QUESTIONNAIRE RESULTS (HOPPE, 1999)**
APPENDIX C	**EXAMPLE OF BRIDGE APPROACH DRAINAGE DETAILS (BRIAUD AT AL., 1997)**
APPENDIX D	**APPROACH SLAB OF HOUSTON, TX**
APPENDIX E	**COMPACTION SPECIFICATION OF HOUSTON, TX**
APPENDIX F	**GUIDE SCHEDULE OF SAMPLE AND TESTING FOR EMBANKMENT**
APPENDIX G	**NUMERICAL SIMULATION RESULTS**
APPENDIX H	**BASIC SOIL TEST**
APPENDIX I	**BEST DEVICE TEST RESULTS**

APPENDIX A EXAMPLE OF APPROACH SLAB DETAILS (BRIAUD ET AL., 1997)

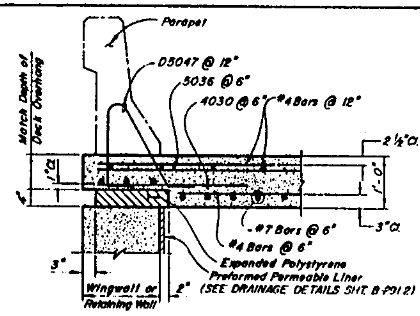
Approach Slab Details





TYPICAL PLAN (NORMAL)
(CONCRETE PAVING)

TYPICAL PLAN (SKEWED)
(CONCRETE PAVING)



SECTION B-B

A) WHEN THE APPROACH SLAB EXTENDS BEYOND THE WINGWALLS, EXTEND THE EXPANDED POLYSTYRENE 2 INCHES BEYOND THE WINGWALL ENDS, ADJUST THE APPROACH SLAB TO ITS FULL DEPTH, AND ELIMINATE THE 5036 BARS.

GENERAL NOTES

1. THE CONCRETE SHALL BE "DA", F'c=4500 PSI, OR "A" F'c=4000 PSI, AS DETERMINED BY THE ENGINEER. WHEN "DA" CONCRETE IS REQUIRED, THE REINFORCING STEEL SHALL HAVE AN EPOXY COATING.

2. A. THE CONTACT JOINT BETWEEN THE CONCRETE PAVEMENT AND THE APPROACH SLAB SHALL PARALLEL THE BACK FACE OF THE STRUCTURE FOR SKEWS OF 20 DEGREES OR LESS; FOR SKEWS GREATER THAN 20 DEGREES THE CONTACT JOINT SHALL BE NORMAL TO THE ROADWAY ALIGNMENT CONTROL LINE. JOINTS SHALL BE STAGGERED ON LANE LINES FOR SKEWED STRUCTURES. STAGGER LINES SHALL BE AT EACH LANE LINE FOR SKEWS OR 45 DEGREES OR MORE.

   B. THE CONTACT JOINT BETWEEN ASPHALT PAVEMENT AND APPROACH SLAB SHALL PARALLEL THE BACK FACE OF THE STRUCTURE.

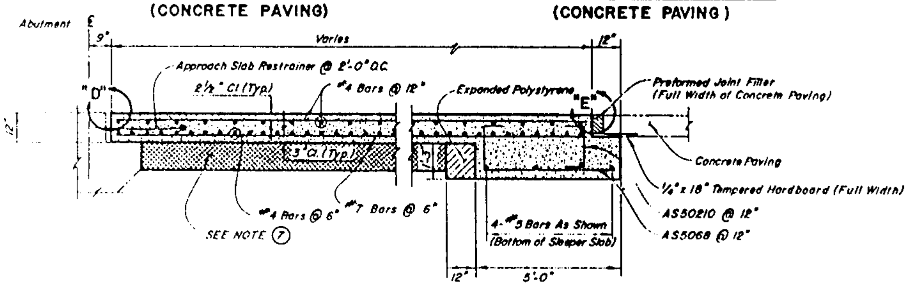
3. FOR SKEWS GREATER THAN 20 DEGREES THE DISTANCE MEASURED NORMAL TO AND FROM THE BACK FACE OF THE STRUCTURE TO THE END OF THE APPROACH SLAB SHALL BE A MINIMUM OR 15 FEET.

4. LONGITUDINAL CONSTRUCTION JOINTS IN THE APPROACH SLAB MAY BE LOCATED ON LANE LINES WHEN PERMITTED BY THE ENGINEER.

5. PLACE 1/4-INCH EXPANSION JOINT MATERIAL BETWEEN THE CONCRETE PAVEMENT AND THE LONGITUDINAL FACE OF THE APPROACH SLAB. THE EXPANSION JOINT MATERIAL IS TO BE RECESSED 1/2-INCH FROM THE SURFACE AND THE JOINT SEALED IDENTICALLY TO THE "LONGITUDINAL WEAKENED PLANE JOINT" ON SHEET R-76 OF THE STANDARD PLANS.

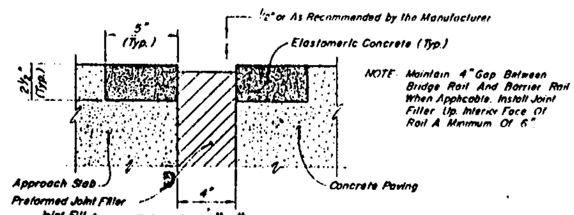
6. THE LENGTH OF THE STEPS MUST BE 12'-0" MINIMUM TO 15'-0" MAXIMUM OR INCREMENTAL INTERVALS (24'-0" MIN. TO 30'-0" MAX...) TO MAINTAIN A 12'-0" MINIMUM TO 15'-0" MAXIMUM SPACING OF THE TRANSVERSE WEAKENED PLANE JOINTS IN THE CONCRETE PAVEMENT. SEE SECTION 409.03.09 OF THE SPECIAL PROVISIONS AND SHEET R-76 OF THE STANDARD PLANS FOR SAW-CUTTING DETAILS.

7. 7. FOR NEW CONSTRUCTION AND REHABILITATION OF EXISTING STRUCTURES, FILL MATERIAL UNDER APPROACH SLABS SHALL BE COMPACTED TO NOT LESS THAN NINETY-FIVE (95) PERCENT OF THE MAXIMUM DENSITY. SEE SECTION 203.03.17 OF THE STANDARD SPECIFICATIONS AND/OR SPECIAL PROVISIONS FOR SPECIFIC TEST METHODS.



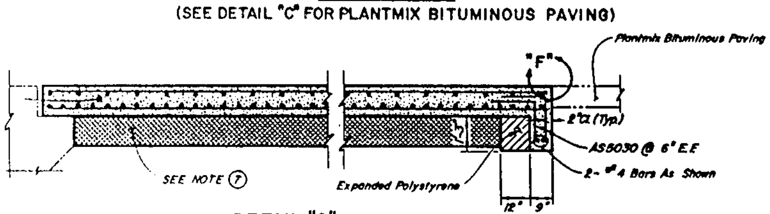
SECTION A-A
(SEE DETAIL "C" FOR PLANTMIX BITUMINOUS PAVING)

SEE CONTRACT PLANS FOR JOINT DETAILS

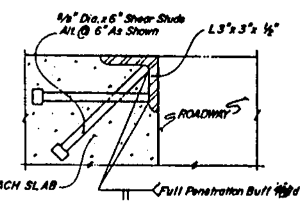


DETAIL "D"
(ELASTOMERIC CONCRETE OPTION)

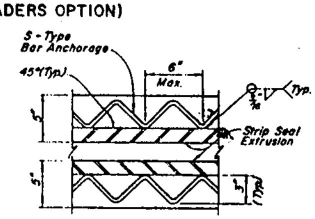
DETAIL "E"
(ELASTOMERIC CONCRETE HEADERS OPTION)



DETAIL "C"
(PLANTMIX BITUMINOUS PAVING)
NOTE: FOR INFORMATION & DIMENSIONS NOT SHOWN SEE SECTION A-A

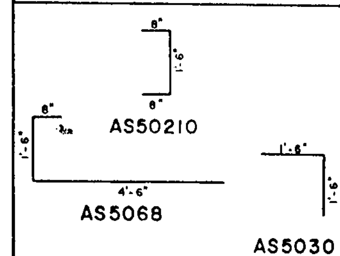


DETAIL "F"
(APPROACH SLAB JOINT PROTECTION-PLANTMIX BITUMINOUS PAVING)



DETAIL "K"
(S BAR ANCHORAGE PLAN)

THIS SHEET IS FOR GENERAL INFORMATION FOR ACTUAL DIMENSIONS AND REINFORCING STEEL LAYOUTS, SEE CONTRACT PLANS.

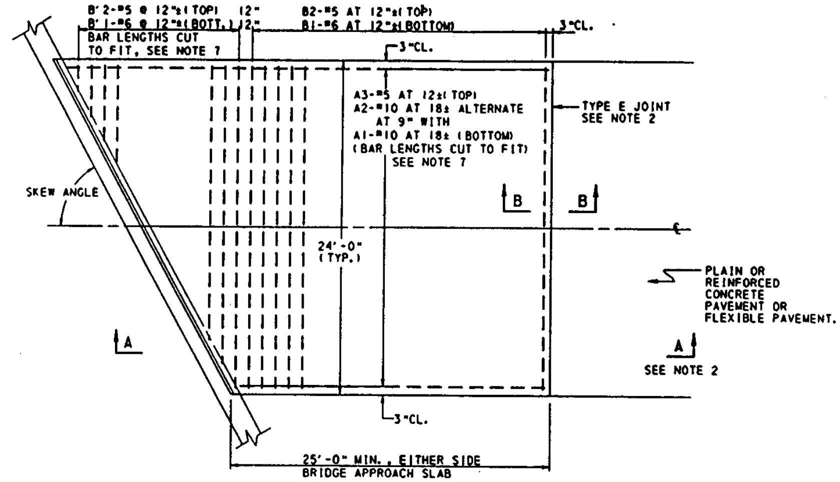


BENT BARS

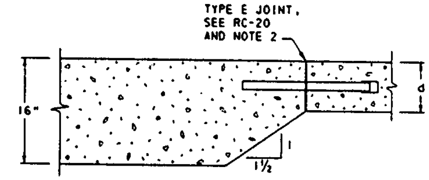
STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION

APPROACH SLABS

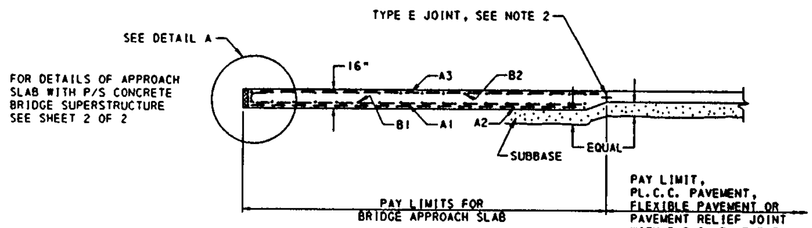
CHIEF BRIDGE ENGR.
B-29.1.1-(502)
ADOPTED: 12/90
REVISION 1-10/92



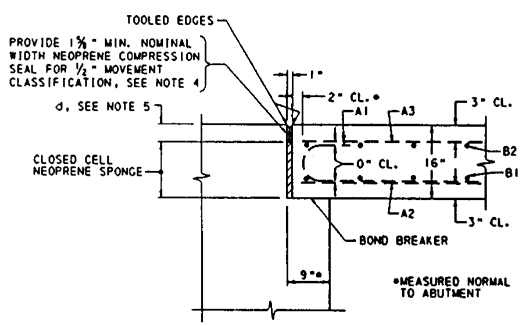
PLAN



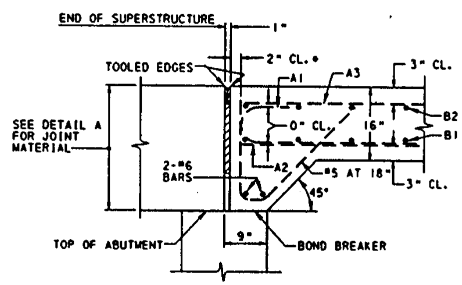
SECTION B-B



SECTION A-A



DETAIL A



DETAIL A (ALTERNATE)

TO APPLY ONLY WHEN INDICATED ON STRUCTURE DRAWINGS

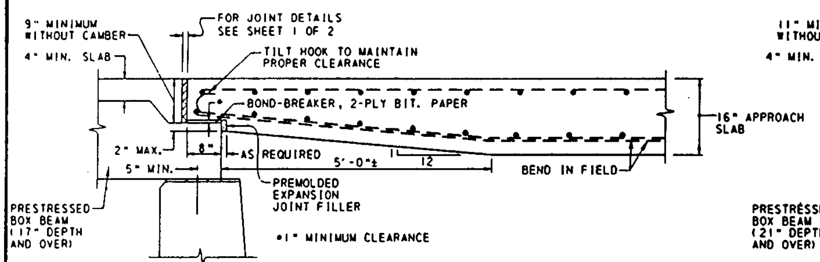
NOTES

1. CONSTRUCT IN ACCORDANCE WITH THIS STANDARD DRAWING OR AS INDICATED ON THE STRUCTURE DRAWINGS.
2. THE TYPE E JOINT DOES NOT APPLY WHEN APPROACH SLAB IS BEING CONSTRUCTED IN CONJUNCTION WITH A PAVEMENT RELIEF JOINT OR WITH A FLEXIBLE PAVEMENT, SEE RC-24.
3. WHEN CONSTRUCTION INVOLVES MORE THAN 2 LANES, CONNECT ADDITIONAL LANES REQUIRED TO STANDARD 2 LANE BRIDGE APPROACH SLAB USING TYPE L CONSTRUCTION JOINTS, AS SHOWN ON RC-20, SHEET 2 OF 2.
4. INSTALL NEOPRENE COMPRESSION SEALS TO A UNIFORM DEPTH WITH TOP OF THE SEAL NOT LESS THAN ¼" NOR MORE THAN ⅜" BELOW THE LEVEL OF THE PAVEMENT SURFACE. THE TOP EDGES OF THE CONTACT SURFACES ON BOTH SIDES OF THE SEAL SHALL BE AT THE SAME ELEVATION.
5. DETERMINE "d" BY ADDING ½" TO THE MAXIMUM COMPRESSED HEIGHT OF THE NEOPRENE COMPRESSION SEAL (SEE MANUFACTURER'S INFORMATION).
6. CONSTRUCT THE BRIDGE APPROACH SLAB AFTER THE BRIDGE DECK IS CONSTRUCTED.
7. PROVIDE REINFORCEMENT BARS, EPOXY COATED IN ACCORDANCE WITH PUBLICATION 408, SECTION 709.

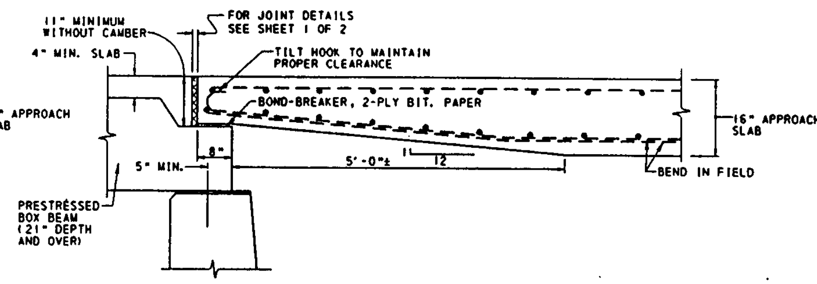
COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION
BUREAU OF DESIGN

BRIDGE APPROACH SLAB

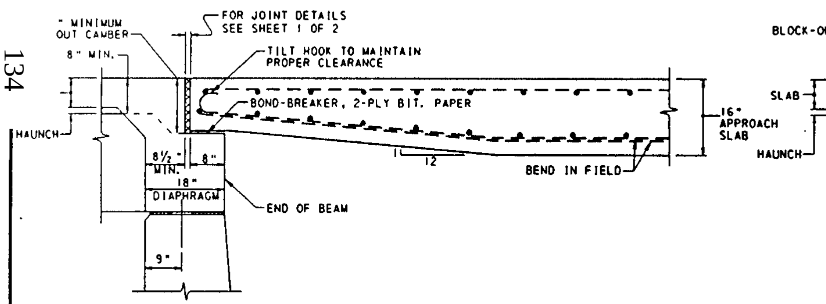
RECOMMENDED MAR. 25, 1994	RECOMMENDED MAR. 25, 1994	SHT. 1 OF 2
DIRECTOR, BUREAU OF DESIGN	CHIEF ENGINEER	RC-23



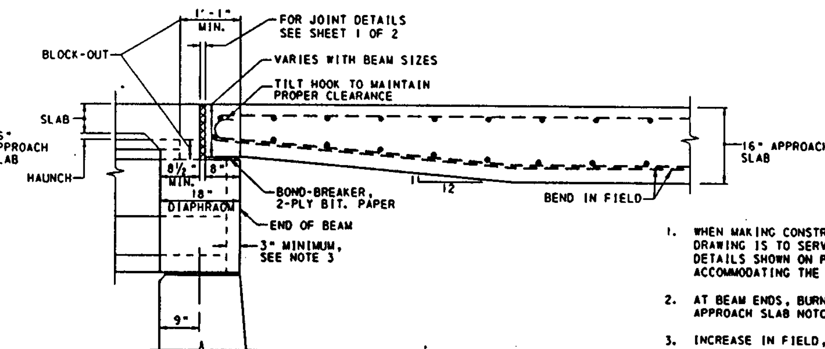
17" DEEP ADJACENT COMPOSITE BOX BEAMS
WITH 9" DEEP APPROACH SLAB NOTCH



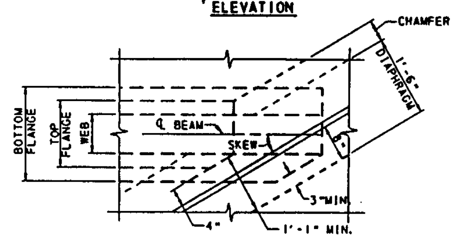
21" TO 48" DEEP ADJACENT COMPOSITE BOX BEAMS
WITH 11" DEEP APPROACH SLAB NOTCH



SPREAD BOX BEAMS WITH APPROACH
SLAB NOTCH 11½" OR DEEPER



ELEVATION



PLAN

I-BEAMS

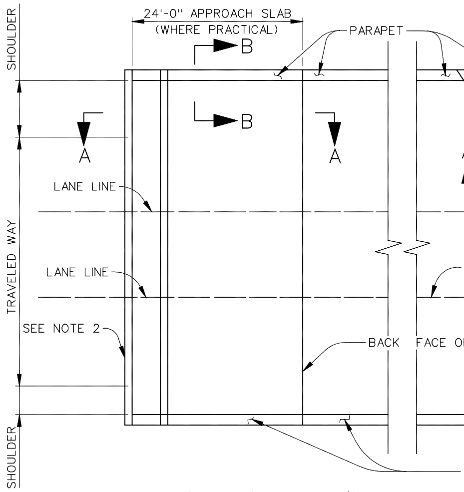
NOTES

1. WHEN MAKING CONSTRUCTION CHANGES IN THE FIELD, THIS DRAWING IS TO SERVE AS A GUIDE FOR MODIFYING NOTCH DETAILS SHOWN ON P/S STANDARD DRAWINGS FOR ACCOMMODATING THE STANDARD 16" BRIDGE APPROACH SLAB.
2. AT BEAM ENDS, BURN OFF REINFORCEMENT PROTRUDING INTO APPROACH SLAB NOTCH.
3. INCREASE IN FIELD, PROVIDING OVERHANG, IF REQUIRED.
4. PROVIDE REINFORCEMENT BARS, EPOXY COATED, IN ACCORDANCE WITH PUBLICATION 408, SECTION 709.

COMMONWEALTH OF PENNSYLVANIA
DEPARTMENT OF TRANSPORTATION
BUREAU OF DESIGN

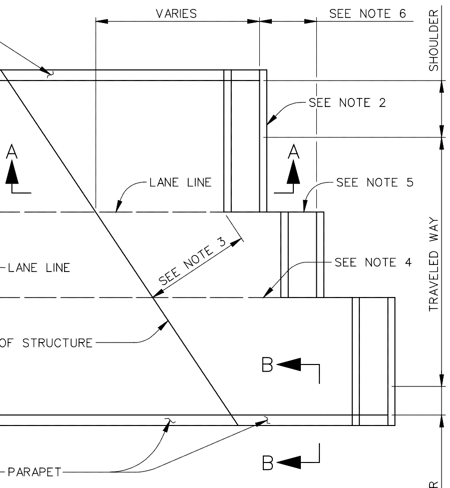
BRIDGE APPROACH SLAB

RECOMMENDED MAR. 25, 1994	RECOMMENDED MAR. 25, 1994	SHT. 2 OF 2
DIRECTOR, BUREAU OF DESIGN	CHIEF ENGINEER	RC-23



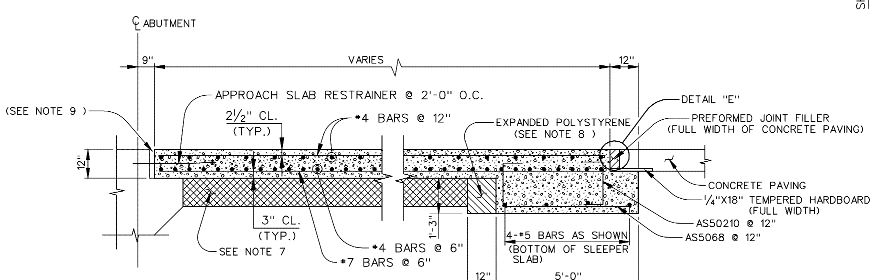
TYPICAL PLAN (NORMAL)

(CONCRETE PAVING)



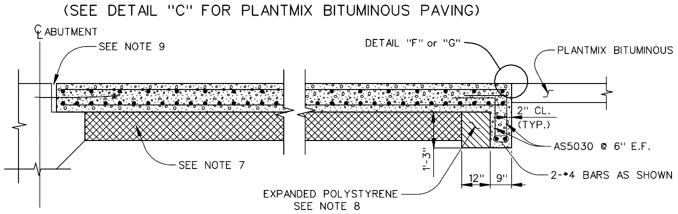
TYPICAL PLAN (SKEWED)

(CONCRETE PAVING)



SECTION A-A

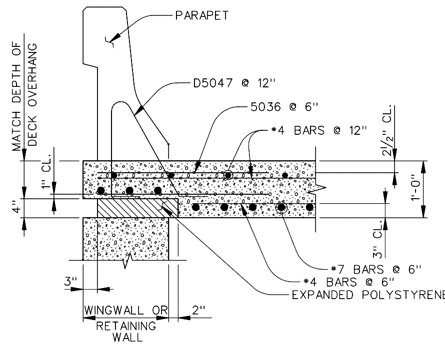
(SEE DETAIL "C" FOR PLANTMIX BITUMINOUS PAVING)



DETAIL "C"

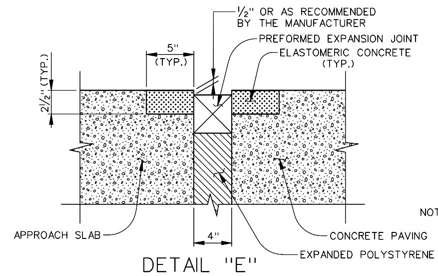
(PLANTMIX BITUMINOUS PAVING)

NOTE: FOR INFORMATION & DIMENSIONS NOT SHOWN SEE SECTION A-A



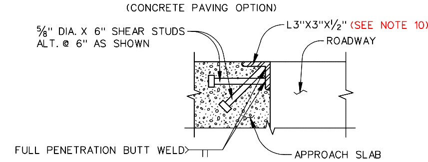
SECTION B-B

A) WHEN THE APPROACH SLAB EXTENDS BEYOND THE WINGWALLS, EXTEND THE EXPANDED POLYSTYRENE 2 INCHES BEYOND THE WINGWALL ENDS. ADJUST THE APPROACH SLAB TO ITS FULL DEPTH, AND ELIMINATE THE 5036 BARS.



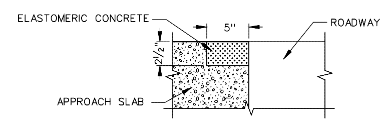
DETAIL "E"

(CONCRETE PAVING OPTION)



DETAIL "F"

(APPROACH SLAB JOINT PROTECTION-PLANTMIX BITUMINOUS PAVING)



DETAIL "G"

(APPROACH SLAB JOINT PROTECTION-PLANTMIX BITUMINOUS PAVING)

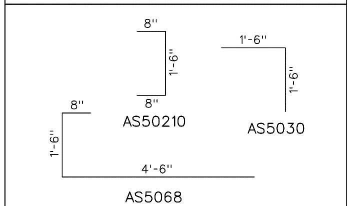
GENERAL NOTES:

1. THE CONCRETE SHALL BE "EA", F'c=4500 PSI, OR "A" F'c=4000 PSI, AS DETERMINED BY THE ENGINEER. WHEN "EA" CONCRETE IS REQUIRED, THE REINFORCING STEEL SHALL HAVE AN EPOXY COATING.
2. A. THE CONTACT JOINT BETWEEN THE CONCRETE PAVEMENT AND THE APPROACH SLAB SHALL PARALLEL THE BACK FACE OF THE STRUCTURE FOR SKEWS OF 20 DEGREES OR LESS; FOR SKEWS GREATER THAN 20 DEGREES THE CONTACT JOINT SHALL BE NORMAL TO THE ROADWAY ALIGNMENT CONTROL LINE. JOINTS SHALL BE STAGGERED ON LANE LINES FOR SKEWED STRUCTURES. STAGGER LINES SHALL BE AT EACH LANE LINE FOR SKEWS OF 45 DEGREES OR MORE.

   B. THE CONTACT JOINT BETWEEN ASPHALT PAVEMENT AND APPROACH SLAB SHALL PARALLEL THE BACK FACE OF THE STRUCTURE.
3. FOR SKEWS GREATER THAN 20 DEGREES THE DISTANCE MEASURED NORMAL TO AND FROM THE BACK FACE OF THE STRUCTURE TO THE END OF THE APPROACH SLAB SHALL BE A MINIMUM OF 15 FEET.
4. LONGITUDINAL CONSTRUCTION JOINTS IN THE APPROACH SLAB MAY BE LOCATED ON LANE LINES WHEN PERMITTED BY THE ENGINEER.
5. PLACE 1/4-INCH EXPANSION JOINT MATERIAL BETWEEN THE CONCRETE PAVEMENT AND THE LONGITUDINAL FACE OF THE APPROACH SLAB. THE EXPANSION JOINT MATERIAL IS TO BE RECESSED 1/2-INCH FROM THE SURFACE AND THE JOINT SEALED IDENTICALLY TO THE "LONGITUDINAL WEAKENED PLANE JOINT" ON SHEET R-76 OF THE STANDARD PLANS.
6. THE LENGTH OF THE STEPS MUST BE 12'-0" MINIMUM TO 15'-0" MAXIMUM OR INCREMENTAL INTERVALS (24'-0" MIN. TO 30'-0" MAX...) TO MAINTAIN A 12'-0" MINIMUM TO 15'-0" MAXIMUM SPACING OF THE TRANSVERSE WEAKENED PLANE JOINTS IN THE CONCRETE PAVEMENT. SEE SECTION 409.03.09 OF THE SPECIAL PROVISIONS AND SHEET R-76 OF THE STANDARD PLANS FOR SAW-CUTTING DETAILS.
7. WHEN CALLED FOR ON THE PLANS, FILL MATERIAL UNDER APPROACH SLABS SHALL BE COMPACTED TO NOT LESS THAN NINETY-FIVE (95) PERCENT OF THE MAXIMUM DENSITY. SEE SECTION 203.03.17 OF THE STANDARD SPECIFICATIONS AND/OR SPECIAL PROVISIONS FOR SPECIFIC TEST METHODS.
8. EXPANDED POLYSTYRENE TO BE USED WHEN NOTED ON THE PLANS.
9. SEE PLANS FOR EXPANSION JOINT DETAILS.
10. GALVANIZE ASSEMBLY AFTER FABRICATION OR USE A588 STEEL. ASSEMBLY TO EXTEND FULL WIDTH OR TO THE FACE OF ANY BRIDGE OR BARRIER RAILS. FULL PENETRATION BUTT WELD ANY FIELD SPLICES.

NOTE: MAINTAIN 4" GAP BETWEEN BRIDGE RAIL AND BARRIER RAIL WHEN APPLICABLE. INSTALL JOINT FILLER UP INTERIOR FACE OF RAIL A MINIMUM OF 6".

THIS SHEET IS FOR GENERAL INFORMATION FOR ACTUAL DIMENSIONS AND REINFORCING STEEL LAYOUTS, SEE CONTRACT PLANS.



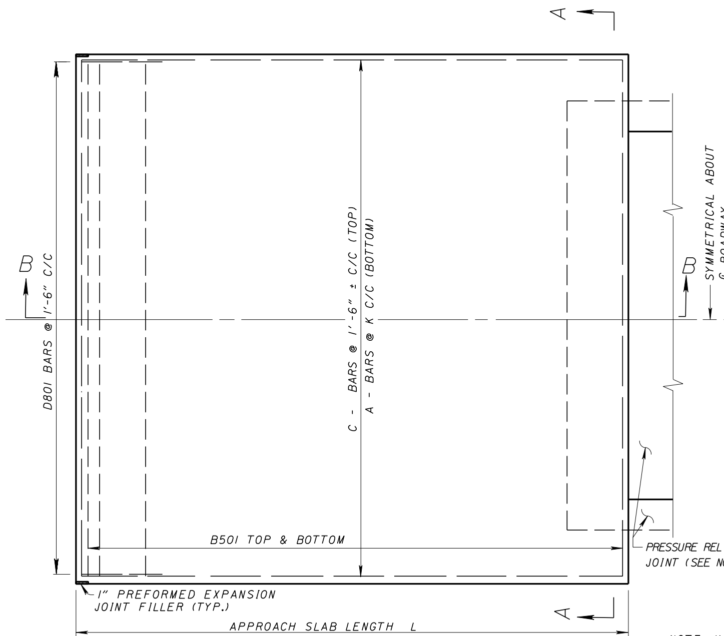
BENT BARS

STATE OF NEVADA
DEPARTMENT OF TRANSPORTATION

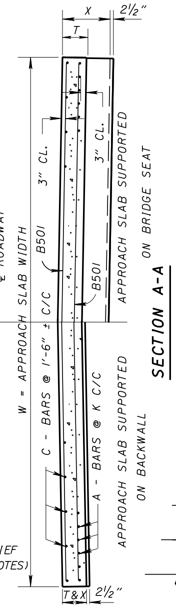
APPROACH SLAB

	B-29.1.1	(502)
CHIEF BRIDGE ENGINEER	ADOPTED: 12/90	REVISION: 10/00

136

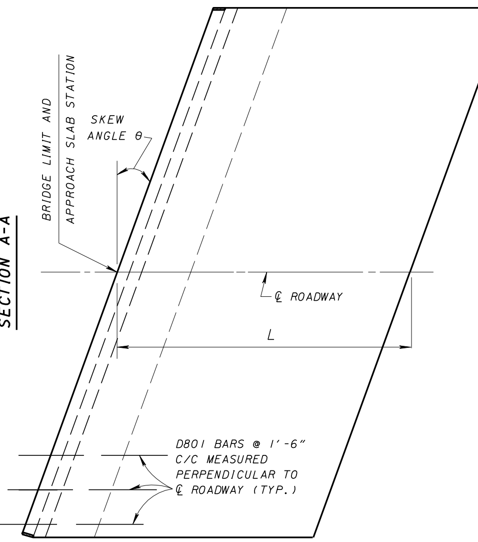


PLAN

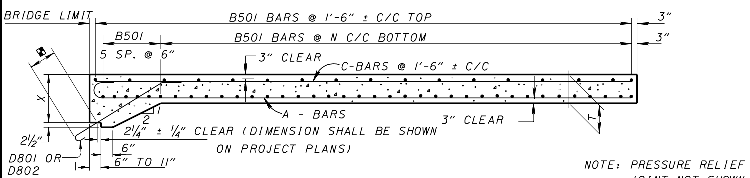


SECTION A-A

NOTE: X SHALL NEVER BE LESS THAN T



APPROACH SLAB FOR SKEWED STRUCTURE

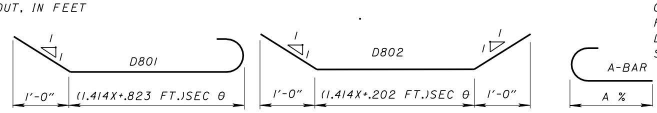


■ = (1'-3") SEC θ (AS MEASURED ALONG THE LENGTH OF THE D801 OR D802 BAR)

SECTION B-B

REINFORCING STEEL (FOR ONE APPROACH SLAB)																
LENGTH L	THICKNESS T	A-BARS					B501 (BOTTOM)			B501 (TOP)		C-BARS			D801 OR D802
		SP'C'G K	MARK	LENGTH	DIMENSION A	NO. REQ'D.	* LENGTH	SP'C'G N	NO. REQ'D.	* LENGTH	NO. REQ'D.	MARK	LENGTH	NO. REQ'D.	NO. REQ'D.
15'-0"	12"	10"	A1001	15'-11"	14'-6"	$12\left[\frac{W-0.5}{K}\right]+1$	(W-0.5)sec θ	9"	22	(W-0.5)sec θ	11	C501	14'-6"	$12\left[\frac{W-0.5}{18}\right]+1$	$12\left[\frac{W-0.5}{18}\right]+1$
20'-0"	13"	7½"	A1002	20'-11"	19'-6"			8"	31		14	C502	19'-6"		
25'-0"	15"	7"	A1003	25'-11"	24'-6"			8"	39		18	C503	24'-6"		
30'-0"	17"	6½"	A1004	30'-11"	29'-6"			8½"	44		21	C504	29'-6"		

W = APPROACH SLAB WIDTH, OUT TO OUT, IN FEET
θ = ANGLE OF SKEW
K = A-BAR SPACING IN INCHES
N = B-BAR SPACING IN INCHES
X = APPROACH SLAB THICKNESS AT ABUTMENT END IN FEET
% = OUT TO OUT



GENERAL: THIS DRAWING PROVIDES DESIGN AND GENERAL CONSTRUCTION DETAILS. THE PROJECT PLANS WILL SHOW LENGTH, SKEW, CURBS (IF ANY), ESTIMATED QUANTITY (SQUARE YARDS), AND SPECIAL NOTES AND DETAILS WHERE NECESSARY. FOR CONDITIONS OTHER THAN THOSE INDICATED HEREON, THE APPROACH SLAB SHALL BE ADAPTED TO FIT THE ENDS OF THE BRIDGE AND THE APPROACH PAVEMENT.

THE D801 OR D802 ANCHOR BARS SHALL BE DIMENSIONED IN THE CONTRACT PLANS AND INCLUDED WITH ITEM 509 FOR PAYMENT FOR EACH SPECIFIC BRIDGE. D801 BARS CANNOT BE USED AS SHOWN IF APPROACH SLABS ARE SUPPORTED ON BACKWALLS LESS THAN 14 INCHES THICK. D802 BARS SHALL BE USED ON PRESTRESSED CONCRETE BOX BEAM BRIDGES WITH APPROACH SLABS SUPPORTED ON BACKWALLS 11 INCHES THICK.

D801 BARS CANNOT BE USED AS SHOWN WHERE APPROACH SLABS ARE SUPPORTED ON BACKWALLS LESS THAN 14 INCHES THICK. D802 BARS SHALL BE USED ON PRESTRESSED CONCRETE BOX BEAM BRIDGES WHERE THE APPROACH SLAB IS SUPPORTED ON AN 11 INCH THICK BACKWALL.

* AT THE OPTION OF THE CONTRACTOR AND AT NO ADDITIONAL COST TO THE STATE, B501 BARS MAY BE LAPPED 2'-6" MINIMUM AT THE CENTERLINE OF ROADWAY, OR WHERE REQUIRED FOR LONGITUDINAL CONSTRUCTION JOINTS, IN LIEU OF PROVIDING FULL LENGTH BARS AS SHOWN.

DESIGN SPECIFICATIONS: THIS STANDARD DRAWING CONFORMS TO "STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES" ADOPTED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS, 1996, INCLUDING THE 1997, 1998 & 1999 INTERIM SPECIFICATIONS AND THE ODOT BRIDGE DESIGN MANUAL.

DESIGN DATA:
DESIGN LOADING: DEAD LOAD - 60 LB/FT^2 (F.W.S.)
LIVE LOAD - HS25 AND THE ALTERNATE MILITARY LOADING.

CONCRETE - COMPRESSIVE STRENGTH = 4500 PSI.

REINFORCING STEEL - MIN. YIELD STRENGTH = 60,000 PSI.

REINFORCING STEEL FOR SKEWED BRIDGES: THE A AND C BARS SHALL BE PLACED PARALLEL TO THE CENTER LINE OF ROADWAY AND THE B BARS SHALL BE PLACED PARALLEL TO THE ABUTMENTS.

LONGITUDINAL CONSTRUCTION JOINTS REQUIRED FOR STAGE CONSTRUCTION SHALL BE ACCORDING TO 511.12.

CURBS, BRIDGES WITH SIDEWALKS: FOR BRIDGES CONSTRUCTED WITH RAISED SIDEWALKS, DEFLECTOR PARAPETS OR OTHER TYPES OF CONSTRUCTION WHICH RETAIN ROADWAY SURFACE DRAINAGE, THE APPROACH SLABS SHALL EITHER INCLUDE INTEGRAL CURBS OR BE CONSTRUCTED IN CONJUNCTION WITH BRIDGE CURBS. CURB HEIGHT SHALL BE TRANSITIONED UNIFORMILY BETWEEN BRIDGE CURB HEIGHT AND ROADWAY CURB HEIGHT IN A LENGTH AS FOLLOWS: WHERE WINGWALL EXTENDS BEYOND END OF APPROACH SLAB, USE A MINIMUM LENGTH OF 10 FEET BEYOND END OF WINGWALL. WHERE THE APPROACH SLAB EXTENDS BEYOND THE END OF WINGWALL,TRANSITION IN THIS LENGTH. HOWEVER, THE TRANSITION LENGTH SHALL NOT BE LESS THAN 10 FEET AND THE TRANSITION SHALL EXTEND BEYOND THE END OF APPROACH SLAB IF NECESSARY.

APPROACH SLAB WIDTH (W): APPROACH SLABS SHALL BE THE SAME WIDTH AS THE BRIDGE ROADWAY.

APPROACH SLAB LENGTH (L): THE LENGTH SHOULD BE BASED ON FACTORS SUCH AS THE SIZE AND AMOUNT OF EXCAVATION BEHIND THE ABUTMENTS, NEW OR EXISTING EMBANKMENTS AND SKEW OF THE BRIDGE. THE LENGTH SHALL BE SHOWN ON THE PROJECT PLANS.

DECK CROWN AND SLOPE: THE LOCATION OF THE CROWN POINT AND THE RATE OF CROSS SLOPE ON THE APPROACH SLAB SHALL CONFORM TO THAT OF THE BRIDGE DECK AND APPROACH PAVEMENT. IF THE RATE OF CROSS SLOPE OF THE BRIDGE DECK DIFFERS FROM THAT OF THE APPROACH PAVEMENT, A SMOOTH TRANSITION SHALL BE PROVIDED WITHIN THE LIMITS OF THE APPROACH SLAB WHENEVER POSSIBLE.

WEARING SURFACE: GENERALLY APPROACH SLABS SHALL HAVE AN ASPHALT CONCRETE WEARING SURFACE ONLY WHEN BOTH THE APPROACH PAVEMENT SURFACE AND THE BRIDGE WEARING SURFACE ARE ASPHALT CONCRETE.

PRESSURE RELIEF JOINTS: RELIEF JOINTS, TYPE A, ARE TO BE PROVIDED REGARDLESS OF ABUTMENT DESIGN AT ALL BRIDGE APPROACHES WHERE APPROACH PAVEMENT IS RIGID, OR COMPOSITE CONSISTING OF A RIGID BASE. SEE STANDARD CONSTRUCTION DRAWING BP-2.3 FOR DETAILS

BASIS OF PAYMENT: IN ADDITION TO THE INCIDENTAL ITEMS LISTED IN 526.08, THE DEPARTMENT WILL INCLUDE THE FOLLOWING ITEMS FOR PAYMENT: THE PREFORMED EXPANSION JOINT FILLER AND JOINT SEALER AT THE CORNERS AND SIDES OF THE APPROACH SLAB; AND THE TYPE "A" WATERPROOFING AND THE PREFORMED ELASTOMERIC COMPRESSION JOINT SEAL AT THE BRIDGE LIMIT END OF THE APPROACH SLAB.

THE DEPARTMENT WILL PAY FOR THE PRESSURE RELIEF JOINTS AND ANCHOR BARS (D801 OR D802) SEPARATELY.

DESIGN AGENCY: OFFICE OF STRUCTURAL ENGINEERING

STATE OF OHIO DEPARTMENT OF TRANSPORTATION

ENGINEER OF BRIDGES — DATE 10-25-94

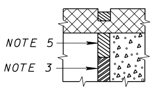
REVIEWED LMW | CHECKED JAM | DESIGNED JFF | DRAWN JFF

AS-1-81

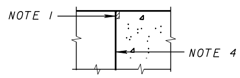
REVISIONS 9-15-94, 04-20-01, 07-19-02

STANDARD: REINFORCED CONCRETE APPROACH SLAB

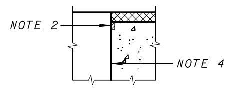
1/3



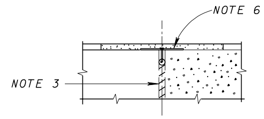
DETAIL A



DETAIL B



DETAIL C



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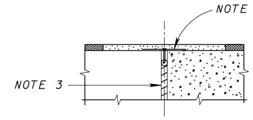
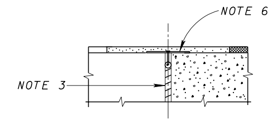
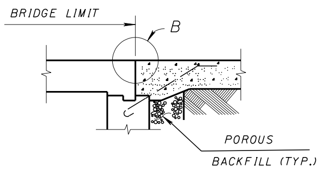


DETAIL E

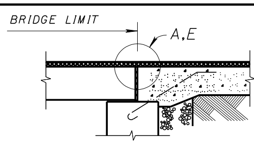


DETAIL F

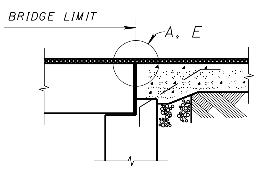


ON SLAB BRIDGES

CONCRETE WEARING
SURFACE ON BRIDGE DECK
AND APPROACH SLAB

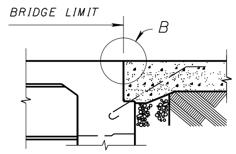


ON PRESTRESSED CONCRETE BOX BEAM BRIDGES ⊗

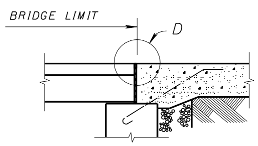


APPROACH SLAB SUPPORTED ON ABUTMENT BACKWALL

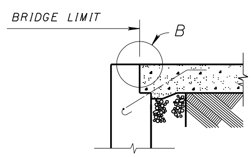
ASPHALT CONCRETE WEARING
SURFACE ON BRIDGE DECK
AND APPROACH SLAB



ON BRIDGES WITH INTEGRAL CONSTRUCTION
(SEMI-INTEGRAL SIMILAR)

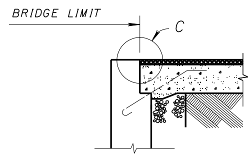


ON PRESTRESSED CONCRETE BOX BEAM BRIDGES ⊗

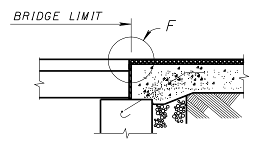


APPROACH SLAB SUPPORTED ON ABUTMENT BACKWALL

CONCRETE WEARING
SURFACE ON BRIDGE DECK
AND APPROACH SLAB



APPROACH SLAB SUPPORTED ON ABUTMENT BACKWALL



ON PRESTRESSED CONCRETE BOX BEAM BRIDGES ⊗

CONCRETE WEARING
SURFACE ON BRIDGE
DECK ONLY

⊗ - THE APPROACH SLAB SEAT FOR THIS PRESTRESSED CONCRETE BOX BEAM BRIDGE IS SHOWN AT THE SAME ELEVATION AS THE BEAM SEAT. HOWEVER, IT MAY ACTUALLY BE HIGHER OR LOWER THAN THE BEAM SEAT DEPENDING ON THE BOX BEAM DEPTH.

NOTE 1: PREFORMED ELASTOMERIC COMPRESSION JOINT SEAL, 705.11 (1¼" WIDE FOR A ½" WIDE GROOVE) PLACED IN ½" x 2¼" GROOVE.

NOTE 2: PREFORMED ELASTOMERIC COMPRESSION JOINT SEAL, 705.11 (1¼" WIDE FOR A ½" WIDE GROOVE) PLACED IN ½" x 2⅛" GROOVE.

NOTE 3: 1" PREFORMED EXPANSION JOINT FILLER, 705.03.

NOTE 4: TYPE "A" WATERPROOFING.

NOTE 5: SEE PLAN INSERT SHEET, "ABUTMENT JOINTS IN BITUMINOUS CONCRETE, BOX BEAM BRIDGES."

NOTE 6: SEE PLAN INSERT SHEET, "POLYMER MODIFIED ASPHALT EXPANSION JOINT SYSTEM."

TYPE "A" WATERPROOFING SHALL NOT EXTEND ABOVE THE BOTTOM OF THE GROOVE INTO WHICH THE PREFORMED ELASTOMERIC COMPRESSION JOINT SEAL IS TO BE PLACED. IT SHALL BE APPLIED TO THE ENTIRE AREA OF THE ABUTMENT OR SUPERSTRUCTURE WHICH COMES INTO CONTACT WITH THE APPROACH SLAB.

FOR PRESTRESSED CONCRETE BOX BEAM BRIDGES WITH ASPHALT CONCRETE ON BOTH BRIDGE DECK AND APPROACH SLAB, THE TOP OF APPROACH SLAB AT THE BRIDGE END SHALL BE CONSTRUCTED TO THE LEVEL OF THE TOP OF THE BEAMS TO FACILITATE WATERPROOFING OF THE JOINT. THE THICKNESS OF ASPHALT CONCRETE AT THE APPROACH END OF THE SLAB SHALL BE THE THICKNESS OF ASPHALT CONCRETE USED ON THE ROADWAY PAVEMENT. THE THICKNESS OF ASPHALT CONCRETE SHALL VARY UNIFORMLY, IF NECESSARY, IN THE LENGTH OF THE APPROACH SLAB. THE SUBGRADE (SUBBASE) SHALL BE GRADED TO PERMIT THE BOTTOM OF THE APROACH SLAB TO BE PARALLEL TO THE TOP.

FOR STRUCTURES WITHOUT STRIP SEAL, COMPRESSION SEAL OR POLYMER MODIFIED ASPHALT EXPANSION JOINTS, THAT HAVE AN ASPHALT CONCRETE WEARING SURFACE ON BOTH THE BRIDGE DECK AND APPROACH SLAB, EXTEND THE DECK WATERPROOFING 2'-0" BEYOND THE BRIDGE LIMITS. FOR STRUCTURES WITH STRIP SEAL AND COMPRESSION SEAL EXPANSION JOINTS, END THE DECK WATERPROOFING AT THE PRESTRESSED BOX BEAM NOTCH. FOR STRUCTURES WITH POLYMER MODIFIED ASPHALT EXPANSION JOINTS, EXTEND THE DECK WATERPROOFING TO THE CENTERLINE OF THE JOINT.

DESIGN AGENCY
OFFICE OF STRUCTURAL ENGINEERING

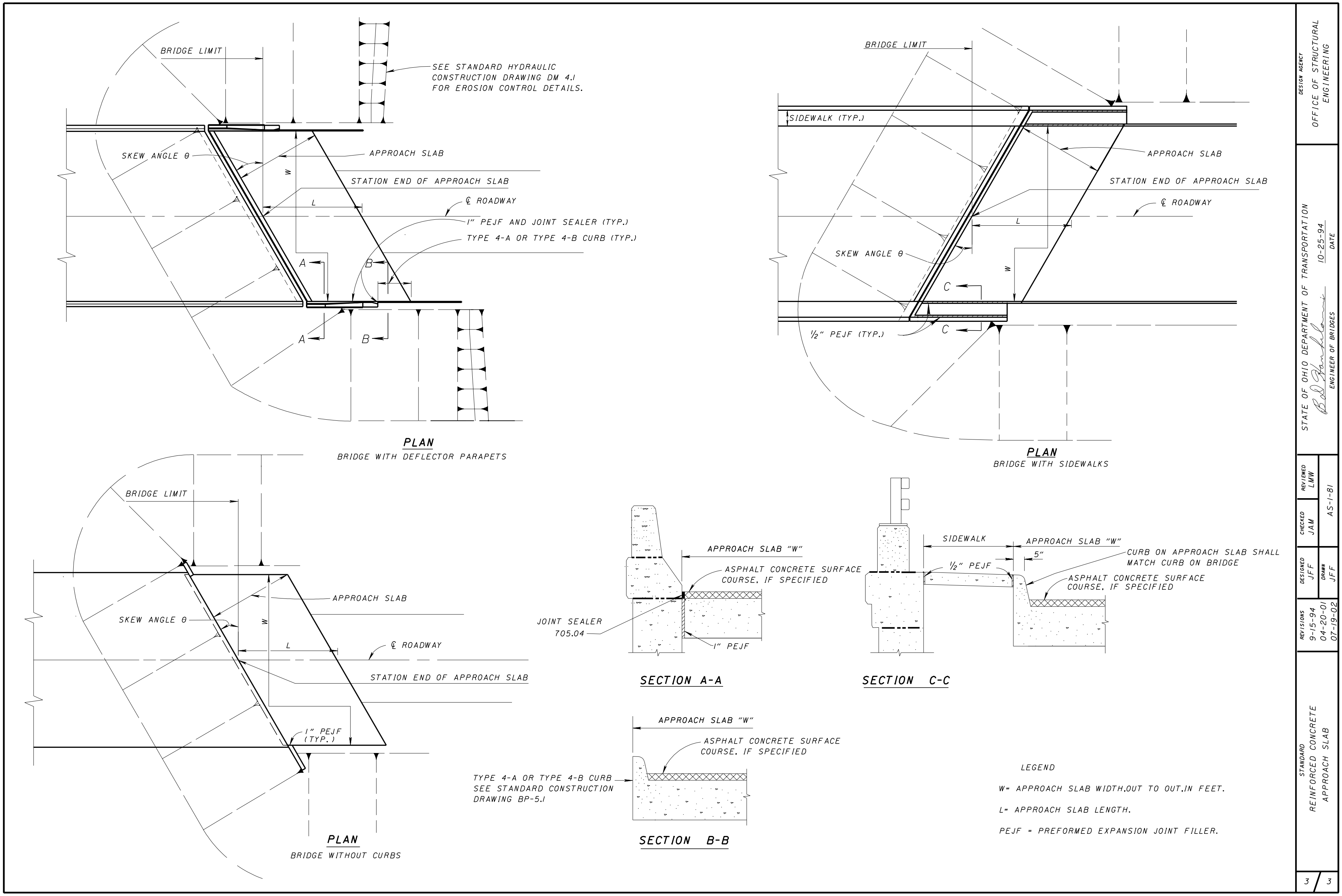
STATE OF OHIO DEPARTMENT OF TRANSPORTATION
10-25-94
DATE
ENGINEER OF BRIDGES

REVIEWED LMW
CHECKED JAM
DESIGNED JFF
DRAWN JFF
AS-1-81

REVISIONS
9-15-94
04-20-01
07-19-02

STANDARD
REINFORCED CONCRETE
APPROACH SLAB

2/3


BRIDGE LIMIT
SEE STANDARD HYDRAULIC CONSTRUCTION DRAWING DM 4.1 FOR EROSION CONTROL DETAILS.
SKEW ANGLE θ
APPROACH SLAB
STATION END OF APPROACH SLAB
℄ ROADWAY
1" PEJF AND JOINT SEALER (TYP.)
TYPE 4-A OR TYPE 4-B CURB (TYP.)
W
L
A
B
PLAN
BRIDGE WITH DEFLECTOR PARAPETS
SIDEWALK (TYP.)
½" PEJF (TYP.)
C
PLAN
BRIDGE WITH SIDEWALKS
1" PEJF (TYP.)
PLAN
BRIDGE WITHOUT CURBS
APPROACH SLAB "W"
ASPHALT CONCRETE SURFACE COURSE, IF SPECIFIED
JOINT SEALER 705.04
1" PEJF
SECTION A-A
SIDEWALK
½" PEJF
5"
CURB ON APPROACH SLAB SHALL MATCH CURB ON BRIDGE
SECTION C-C
TYPE 4-A OR TYPE 4-B CURB SEE STANDARD CONSTRUCTION DRAWING BP-5.1
SECTION B-B
LEGEND
W= APPROACH SLAB WIDTH,OUT TO OUT,IN FEET.
L= APPROACH SLAB LENGTH.
PEJF = PREFORMED EXPANSION JOINT FILLER.
DESIGN AGENCY
OFFICE OF STRUCTURAL ENGINEERING
STATE OF OHIO DEPARTMENT OF TRANSPORTATION
10-25-94
DATE
ENGINEER OF BRIDGES
REVIEWED LMW
CHECKED JAM
DESIGNED JFF
DRAWN JFF
AS-1-81
REVISIONS 9-15-94 04-20-01 07-19-02
STANDARD
REINFORCED CONCRETE APPROACH SLAB
3 / 3

APPENDIX B QUESTIONNAIRE RESULTS (HOPPE, 1999)

Table B-1. Advantages of Using Approach Slabs.

State	Smooth Ride	Reduced Impact	Control Drainage	Uniform Settlement	Lower Maint. Cost	Seismic Stability	Minimum Deviation at Joints	None
AL	X	X						
AZ	X	X						
CA	X							
CT	X							
DE	X							
FL	X							
GA	X							
ID		X		X				
IL			X	X				
IN	X			X				
IO	X	X					X	
KS	X	X	X					
KY								X
LA		X						
ME	X	X		X				
MD								X
MA	X							
MN	X	X						
MS	X							
MO	X					X		
MT	X	X						
NE	X		X	X	X			
NH				X				
NJ	X	X						
NM	X							
NY	X							
ND	X				X			
OH	X							
OK	X							
OR	X		X	X		X		
SD	X	X	X					
TX	X							
VT	X	X						
VA	X	X		X				
WA	X					X		
WI	X	X			X			
WY		X	X	X				

Table B-2. Disadvantages of Using Approach Slabs.

State	Higher Initial Cost	Maint.	Erosion	Bending Stress at Backwall	Problems w/Staged Construction	Joints	Rough Surface	Increased Construction Time
CA	X							
DE	X	X	X					
GA		X	X					
IL	X							
IN	X							
IO	X	X						
KS	X	X						
KY	X	X						
LA				X				
ME	X							
MN		X						
MO	X							
MT		X	X			X		
NE	X	X						
NJ		X						
ND	X							
OK	X							X
OR	X						X	X
SD	X	X						
VA		X	X					
WA	X				X			
WI	X	X						
WY	X							

Table B-3. Current Use of Approach Slabs (%).

State	Interstate System	Primary System	Secondary System
AL	100	100	20
AZ	100	100	80
CT	< 50	< 50	< 50
DE	90	65	20
FL	100	100	100
GA	100	100	100
ID	“small”	“small”	“very small”
IL	100	100	90
IN	100	100	100
IA	100	75	10
KS	90	50	20
KY	35	35	35
LA	100	100	100
ME	>50	>50	>50
MD	<1	<2	0
MA	100	100	100
MN	90	69	8
MO	100	100	10
MS	100	100	85
MT	<5	<5	<1
NE	100	100	100
NV	100	100	100
NH	95	30	7
NM	80	80	80
NY	100	100	100
ND	75	60	0
OH	100	95	75
OK	100	>90	0
OR	100	100	100
SC	100	100	30
SD	95	90	5
VT	100	100	100
VA	98	75	<4
WA	75	50	25
WI	100	100	25
WY	90	75	50

Table B-4. Criteria for Use of Approach Slabs with Conventional Abutments.

State	Use on All Bridges	ADT, AADT, DHV	Pavement Type	Settlement Expected	Road Functional Classification	Embankment Height	Engineer's Option	Not Used
AL		X			X			
AZ	X							
CA			X		X			
CT			X	X				
DE		X		X	X	X		
FL	X							
GA	X							
ID			X					
IL					X			
IN	X							
IA		X	X	X	X			
KS		X	X					
KY								X
MA	X							
MD								X
ME		X		X	X	X		
MN					X			
MS	X							
MO	X							
MT		X	X					
NE	X							
NH	X							
NJ		X			X			
NM	X							
NV	X							
NY	X							
ND					X			
OH				X				
OK					X			
OR		X						
SC		X				X		
SD					X			
TX							X	
VT	X							
VA		X			X			
WA				X				
WI					X			
WY	X							

Table B-5. Criteria for Use of Approach Slabs with Integral Abutments

State	Use on All Bridges	ADT, AADT, DHV	Pavement Type	Settlement Expected	Road Functional Classification	Embankment Height	Engineer's Option	Not Used
AL								X
AZ	X							
CO	X							
CT			X	X				
DE								X
FL								X
GA	X							
ID	X							
IL	X							
IN	X							
IA		X	X	X	X			
KS		X	X	X				
KY	X							
MA	X							
MD								X
ME		X		X		X		
MN					X			
MS								X
MO	X							
MT		X	X					
NE	X							
NH	X							
NJ								X
NM	X							
NV	X							
NY	X							
ND					X			
OK	X							
OR		X						
SC		X				X		
SD					X			
TX								X
VT	X							
VA	X							
WA				X				
WI					X			
WY	X							

Table B-6. Special Inclusion Circumstances.

State	Skew	Expected Settlement	Road Class	Engineer's Option	Traffic Volume	Span Length	Pavement Type	Seismic Stability	All Bridges	None
AL				X						
AZ										X
CT										X
FL									X	
GA									X	
ID	X	X								
IL										X
IN									X	
IO		X								
KS				X						
KY			X		X	X				
MA										X
ME		X								
MN		X			X					
MS										X
MO										X
MT		X			X					
NE									X	
NH									X	
NJ		X								
NY									X	
ND										X
OH									X	
OK					X					
OR		X								
SC										X
SD			X							
TX										X
VT					X		X			
VA		X	X	X	X					
WA								X		
WI				X						
WY										X

Table B-7. Special Exclusion Circumstances.

State	No Settle. Expected	Excessive Settle. Expected	Engineer's Option	Traffic Volume	Existing Embank.	Span Length	Pvmt. Type	Rocky Terrain	Retro-fit	None
AL				X						
AZ										X
CT										X
DE						X				
FL										X
GA										X
ID										X
IL										X
IN										X
IO	X			X						
KS										X
KY	X									
ME	X									
MA										X
MS										X
MT										X
NE	X									
NV									X	
NH										X
NJ	X							X		
NY										X
ND				X						
OH					X					
OK										X
OR	X									
SC	X			X						
SD							X			
TX										X
VT				X			X			
VA				X						
WA		X								
WI			X							
WY										X

Table B-8. Typical Approach Slab Dimensions.

State	Length, m (ft)	Thickness, mm (in)	Width	Miscellaneous
AL	6.1 (20)	230 (9)	Pavement	
AZ	4.6 (15)			
CA	3.0-9.1 (10-30)	305 (12)	Curb-Curb	
DE	5.5-9.1 (18-30)			
FL	6.1 (20)	305 (12)	Curb-Curb	
GA	6.1-9.1 (20-30)	254 (10)	Curb-Curb	
ID	6.1 (20)	305 (12)		Length varies with skew angle
IL	9.1 (30)	380 (15)	Curb-Curb	
IN	6.2 (20.5)			Length varies with skew angle
IO	6.1 (20)	254-305 (10-12)	Pavement	Length varies with skew angle
KS	4.0 (13)	254 (10)	Curb-Curb	
KY	7.6 (25)		Curb-Curb	
LA	12.2 (40)	405 (16)	Curb-Curb	Length varies with skew angle
ME	4.6 (15)	203 (8)	Curb-Curb	
MA		254 (10)		Slab is sloped longitudinally
MN	6.1 (20)	305 (12)	Pavement	T-beams
MS	6.1 (20)		Curb-Curb	
MO	7.6 (25)	305 (12)		Timber header at sleeper slab
NV	7.3 (24)	305 (12)	Curb-Curb	
NH	6.1 (20)	380 (15)		
NJ	7.6 (25)	457 (18)		Used with transition slab 9.1 m × 230-457 mm (30 ft × 9-18 in)
NM	4.6 (15)		Curb-Curb	
NY	3.0-7.6 (10-25)	305 (12)	Curb-Curb	Sleeper slab, length varies with abutment type
ND	6.1 (20)	356 (14)	Curb-Curb	
OH	4.6-9.1 (15-30)	305-432 (12-17)		Length varies with embankment and skew angle
OK	9.1 (30)	330 (13)	Curb-Curb	
OR	6.1-9.1 (20-30)	305-356 (12-14)	Curb-Curb	Length varies with fill height and skew angle
SC	6.1 (20)			
SD	6.1 (20)	230 (9)		
TX	6.1 (20)	254 (10)		
VT	6.1 (20)			
VA	6.1-8.5 (20-28)	380 (15)	Pavement	Length varies with skew angle
WA	7.6 (25)	330 (13)	Pavement	Length varies with skew angle
WI	6.2 (20.5)	305 (12)		
WY	7.6 (25)	330 (13)	Curb-Curb	Sleeper slab 1.7 m × 254 mm (5.5 ft × 10 in)

Table B-9. Slab to Backwall Connection.

State	Conventional Bridges		Integral Bridges		Integral Abutments Not Used
	Doweled or Tied	No Connection	Doweled or Tied	No Connection	
AL	X				X
AZ		X			
CA	X		X		
CT		X			
DE		X			X
FL	X				X
GA		X			
ID	X		X		
IL	X		X		
IN		X	X		
IA	X			X	
KS	X		X		
KY		X			
LA	X				
ME		X	X		
MD					X
MA	X			X	
MN		X	X		
MO	X				
MS		X			X
MT		X			
NV	X			X	
NH	X				
NJ		X			X
NM	X				
NY		X			
ND		X		X	
OH	X				
OK	X		X		
OR	X		X		
SC	X				
SD		X		X	
TX	X				X
VT	X				
VA		X	X		
WA	X		X		
WI		X			
WY	X		X		

Table B-10. Embankment Material Specifications.

State	Same/Different from Regular Embankment	% Passing 75 µm (No. 200) Sieve	Miscellaneous
AL	Same		A-1 to A-7
AZ	Different		
CA		<4	Compacted pervious material
CT	Different	<5	Pervious material
DE	Different		Borrow type C
FL	Same		A-1,A-2-4 through A-2-7,A-4,A-5,A-6,A-7 (LL<50)
GA	Same		GA Class I, II or III
ID			A yielding material
IL	Different		Porous, granular
IN	Different	<8	
IO	Different		Granular; can use Geogrid
KS			Can use granular, flowable or lightweight
KY		<10	Granular
LA			Granular
ME	Different	<20	Granular borrow
MA	Different	<10	Gravel Borrow type B, M1.03.0
MI	Different*	<7	*Only top 0.9 m (3 ft) are different (granular materials Class II)
MN		<10	Fairly clean granular
MS	Different		Sandy or loamy, non-plastic
MO			Approved material
MT	Different	<4	Pervious
NE			Granular
NV	Different		Granular
NH	Same	<12	
NJ	Different	<8	Porous fill (Soil Aggregate I-9)
NM	Same		
NY		<15	<30% Magnesium Sulfate loss
ND	Different		Graded mix of gravel and sand
OH	Same		Can use granular material
OK	Different*		*Granular just next to backwall
OR	Different		Better materials
SC	Same		
SD	Varies*		*Different for integral; same for conventional
TX	Same		
VT	Same		Granular
VA	Same		Porous backfill
WA			Gravel borrow
WI	Different	<15	Granular
WY	Different		Fabric reinforced

Table B-11. Lift Thickness and Percent Compaction Requirements.

State	Lift Thickness, mm (in)	% Compaction	Miscellaneous
AL	203 (8)	95	
AZ	203 (8)	100	
CA	203 (8)	95*	*For top 0.76 m (2.5 ft)
CT	152 (6)*	100	*Compacted lift indicated
DE	203 (8)	95	
FL	203 (8)	100	
GA		100	
ID	203 (8)	95	
IL	203 (8)	95*	*For top, remainder varies with embankment height
IN	203 (8)	95	
IO	203 (8)	None	One roller pass per inch thickness
KS	203 (8)	90	
KY	152 (6)*	95	*Compacted lift indicated; Moisture = +2% or -4% of optimum
LA	305 (12)	95	
ME	203 (8)		At or near optimum moisture
MD	152 (6)	97*	*For top 0.30 m (1 ft), remainder is 92%
MA	152 (6)	95	
MI	230 (9)	95	
MN	203 (8)	95	
MS	203 (8)		
MO	203 (8)	95	
MT	152 (6)	95	At or near optimum moisture
NE		95	
NV		95	
NH	305 (12)	98	
NJ	305 (12)	95	
NY	152 (6)*	95	*Compacted lift indicated
ND	152 (6)		
OH	152 (6)		
OK	152 (6)	95	
OR	203 (8)	95*	*For top 0.91 m (3 ft), remainder is 90%
SC	203 (8)	95	
SD	203-305 (8-12)*	97	*0.20 m (8 in) for embankments, 0.30 m (12 in) for bridge end backfill
TX	305 (12)	None	
VT	203 (8)	90	
VA	203 (8)	95	+ or –20% of optimum moisture
WA	102 (4)*	95	*Top 0.61 m (2 ft), remainder is 0.20 m (8 in)
WI	203 (8)	95*	*Top 1.82 m (6 ft and within 60 m (200 ft)), remainder is 90%
WY	305 (12)		Use reinforced geotextile layers

Table B-12. Drainage Provisions.

State	Plastic Pipe	Weep holes	Joint Seal	Granular Fill	Miscellaneous
AL					Open joint on bridge side of abutment
AZ					Geocomposite
CA	X				Filter fabric; geocomposite
CT		X*			*or 152 mm (6 in) underdrain
DE	X	X			
FL			X		Divert water from abutment
GA			X*		*or curb and gutter
ID				X	
IL	X		X		76 × 127 mm (3 × 5 in) curb; can use inlet box
IN	X			X	
IO	X		X		Subdrain at bottom of fill
KS	X		X		Filter fabric and strip drain
KY	X			X	
LA					Wedge of drainable material
ME		X			French drains at abutment and wingwalls
MA					Box culvert, curb, waterproofing
MI	X			X	Underdrain at top of footing
MN				X	Curb and gutter, underdrain at top of footing
MS					No special provisions
MO	X*				*or steel pipe; geotextile fabric
MT	X				Geocomposite
NE					Drainage matting; rock riprap
NH		X*		X	*102 mm (4 in) in diameter
NJ	X*	X		X	*or steel pipe
NM					No special provisions
NY				X	Drainage board
ND	X*				*if soil heave is expected; trench at bottom of backfill.
OH	X			X*	*0.61 m (2 in) thick; underdrain
OK					Underdrain at back of bridge seat
OR			X		End panels; catch basin
SC		X			Geotextile fabric and drains
SD	X				Drainage fabric and waterproofing
TX					No special provisions
VT					No special provisions
VA	X	X		X	
WA					Catch basins and deck grading
WI				X	Underdrains if impervious soil
WY	X				Drainage and filtration geotextile

Table B-13. Construction Issues.

State	Contractors Closely Monitored?	Difficulties Obtaining Specified Degree of Compaction at Abutments?	Recycled or Manufactured Materials Ever Used for Backfilling Abutments?
AL	X	X	
AZ	X		
CA	X		
CT	X	X	X
DE	X		
FL	X		
GA	X	X	
ID	X		
IN	X		
IA	X		
KS		X	X
KY	X	X	X
LA	X	X	X
MA		X	
MD			
ME	X		X
MI	X		X
MS		X	
MO	X		
MT		X	
NE		X	
NH	X	X	
NJ	X	X	
NM	X		
NY	X		
OH		X	
OK		X	
OR	X	X	X
SC	X		X
SD	X	X	X
TX	X	X	
VT	X		
VA	X	X	X
WA	X		X
WI	X		
WY	X	X	

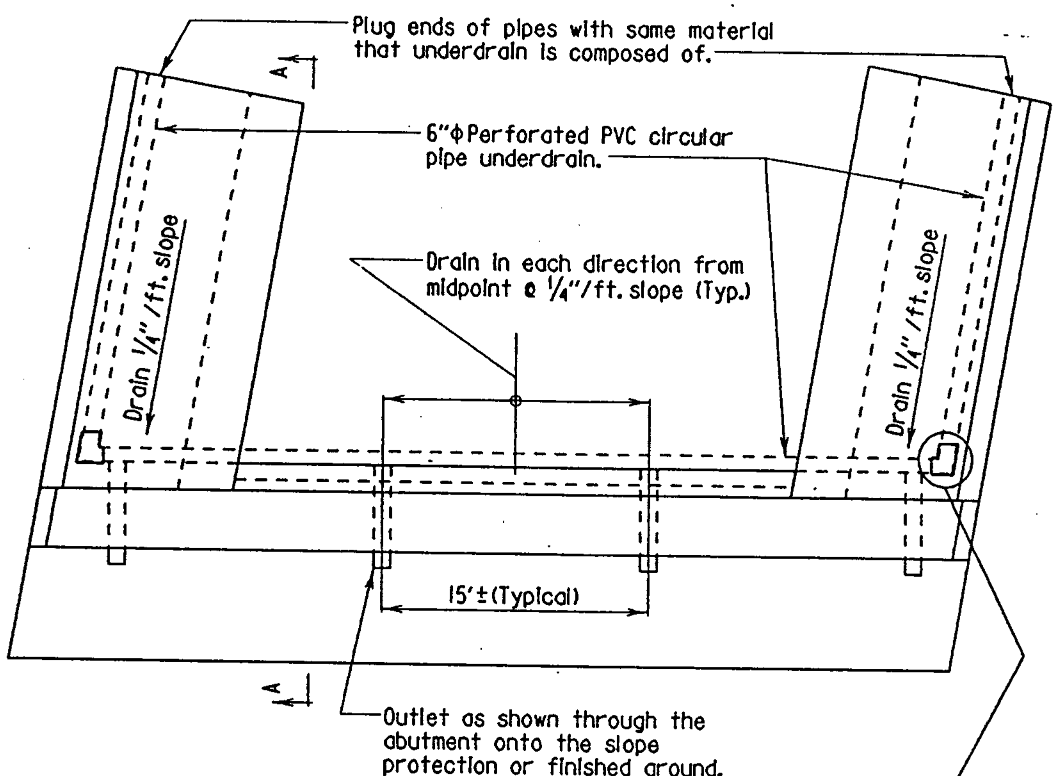
Table B-14. Do You Typically Build Approach Embankments Before or After Abutment Construction?

State	Before	After
AL	X	
AZ		X
CA	X	
CT	X	
DE		X
FL		X
GA	X	X
ID	X	
IN	X	
IA	X	
IL	X	
KS	X	
KY	X	
LA	X	
MA		X
MD		X
ME		X
MI	X	X
MS	X	X
MO		X
MT	X	X
ND	X	
NE	X	X
NH		X
NJ		X
NM	X	
NY	X	X
OH	X	
OK	X	
OR	X	
SC	X	
SD	X	
TX	X	
VT		X
VA	X	X
WA	X	X
WI	X	
WY	X	

Table B-15. Is Approach Slab Settlement a Significant Problem?

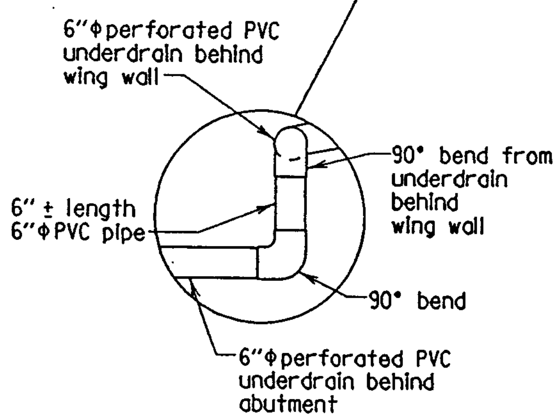
State	Yes	No	Moderate
AZ		X	
CA	X		
CT			X
DE	X		
FL			X
GA	X		
ID	X		
IN			X
IA			X
IL	X		
KS	X		
KY	X		
LA	X		
MA			X
MD			X
ME		X	
MI			X
MN	X		
MS	X		
MO	X		
MT	X		
ND	X		
NE	X		
NH		X	
NJ			X
NM	X		
NY			X
OH			X
OK	X		
OR	X		
SC	X		
SD	X		
TX		X	
VT		X	
VA			X
WA	X		
WI	X		
WY		X	

APPENDIX C EXAMPLE OF BRIDGE APPROACH DRAINAGE DETAILS (BRIAUD ET AL., 1997)



PLAN

Scale: None



Notes:
1. To be used for all abutments, regardless of the direction of the approach roadway grades.
2. Minimum slope of Pipe Underdrain Outlets is 1/4"/ft.
3. For Section A-A see Sheet No. 2, 3 or 4
4. The drainage system behind each wing wall can be connected to the drainage system behind the abutment using 2 - 90° bends and a short length of pipe. This will necessitate the drainage system behind the wing wall be slightly higher.
5. For wing walls over 25 ft. long, the drainage system behind them may be independent of the drainage system behind the abutment. They can be outletted directly through the wing wall.

FHWA APPROVAL
DATE: 6-20-80

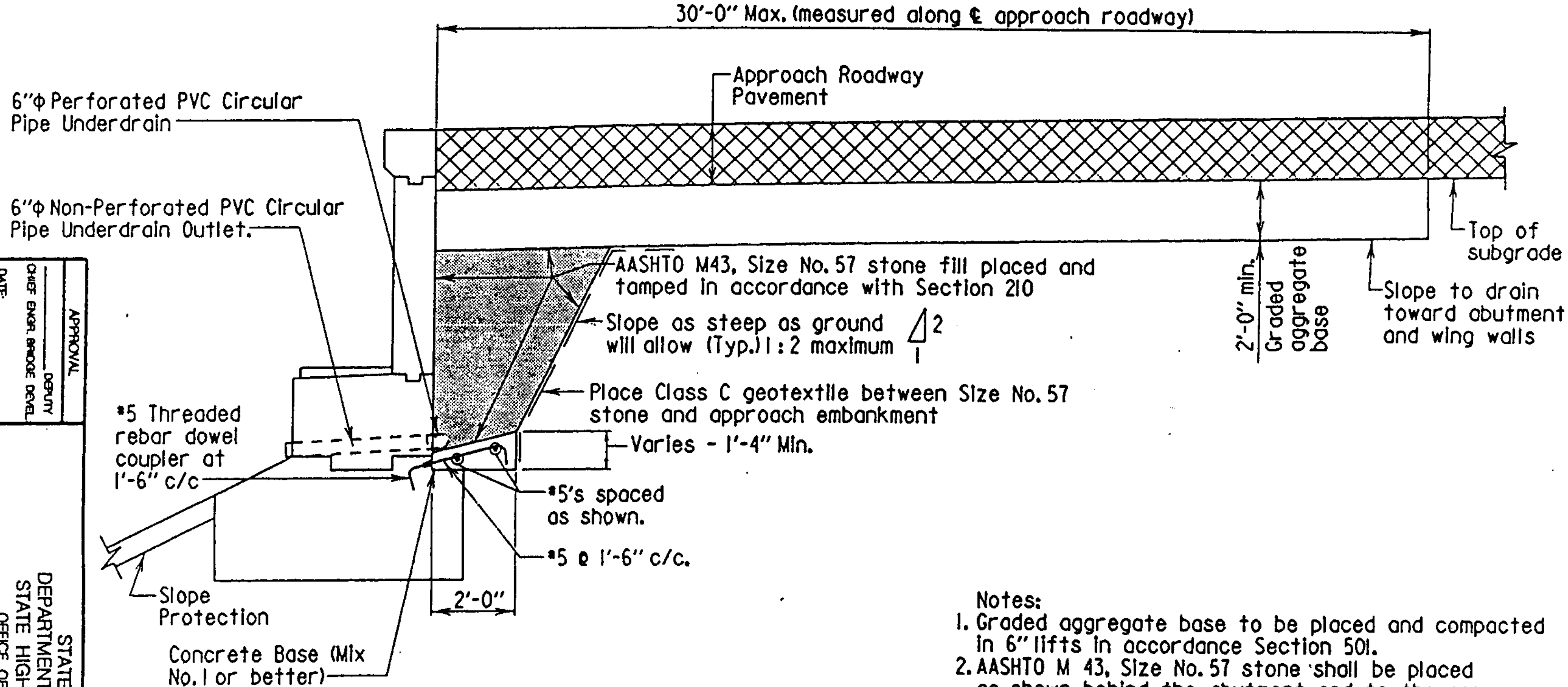
APPROVAL
DEPUTY CHIEF ENGR. BRIDGE DEVEL.
DATE: 3/4/80

REVISIONS	
SHA	FHWA
9-1-82	6-8-90
6-24-87	6-8-90
2-19-92	
2-28-95	

STATE OF MARYLAND
DEPARTMENT OF TRANSPORTATION
STATE HIGHWAY ADMINISTRATION
OFFICE OF BRIDGE DEVELOPMENT

DRAINAGE SYSTEM AND BACKFILL FOR ABUTMENTS

STANDARD NO. BR-SB(0.01)-80-101 SHEET 1 OF 4



SECTION A-A

Scale: None

Notes:

1. Graded aggregate base to be placed and compacted in 6" lifts in accordance Section 501.
2. AASHTO M 43, Size No. 57 stone shall be placed as shown behind the abutment and to the same geometrics behind the abutment wingwalls when they parallel the highway.
3. The Contractor must provide a well compacted surface to place the geotextile and stone fill against. In fill areas it will be necessary to overfill a sufficient distance so that a well compacted surface will be produced when the overfill is removed. The removed material will be paid for as Class 2 Excavation. The maximum pay limits for the stone will be the 2 ft. base and the slope as steep as ground will allow with the 1 : 2 maximum slope.
4. The cost of the stone and geotextile will be incidental to the pertinent structure concrete item.

FHWA APPROVAL

DATE:

APPROVAL

DEPUTY CHIEF ENGR. BRIDGE DEVEL.

DATE:

REVISIONS

SHA | FHWA

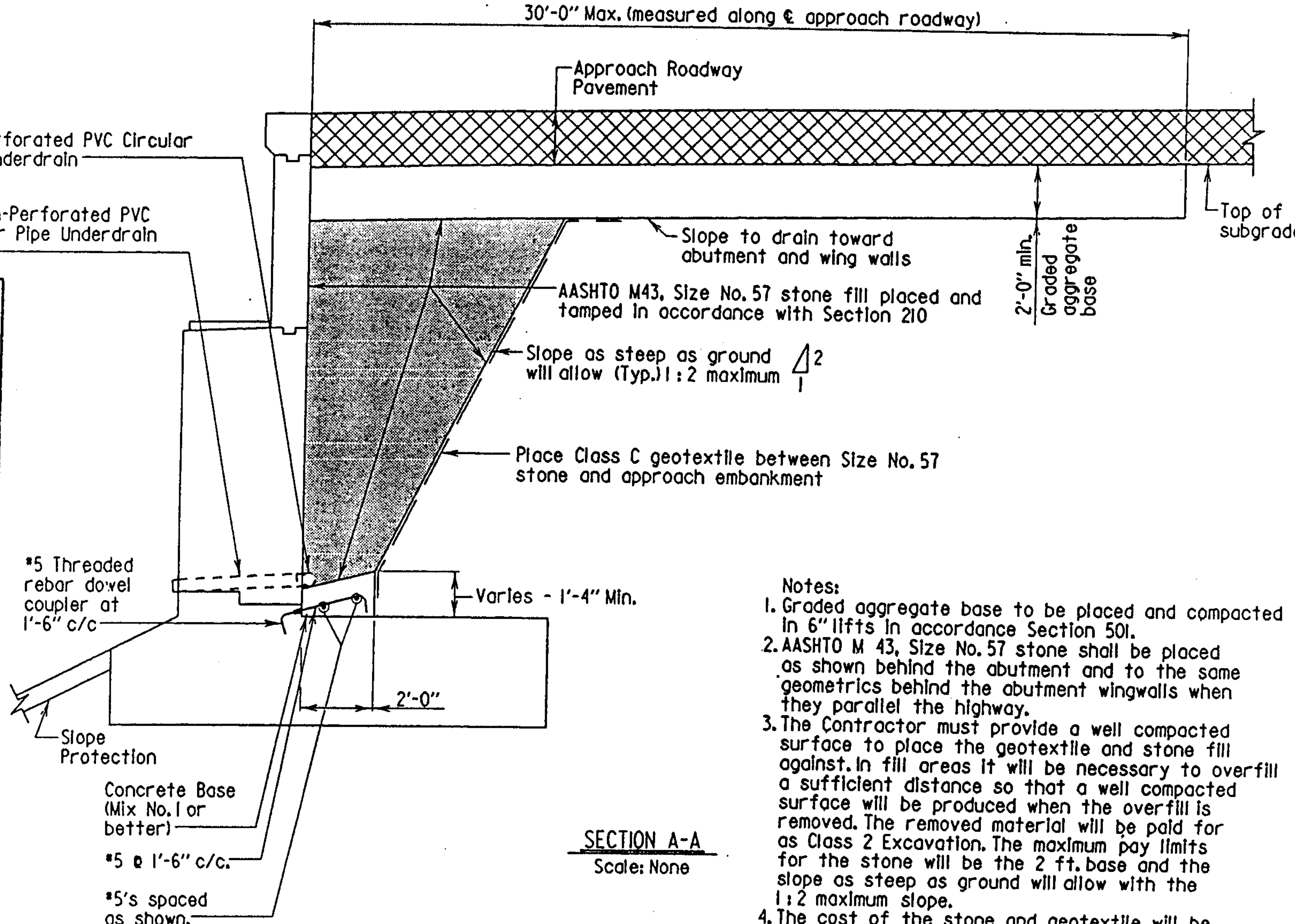
STATE OF MARYLAND
DEPARTMENT OF TRANSPORTATION
STATE HIGHWAY ADMINISTRATION
OFFICE OF BRIDGE DEVELOPMENT

DRAINAGE SYSTEM AND BACKFILL FOR ABUTMENTS (PEDESTAL)

STANDARD NO. BR-SB(0.01)-80-101

SHEET 2 OF 4

SUBSTRUCTURE ABUT



SECTION A-A

Scale: None

Notes:

1. Graded aggregate base to be placed and compacted in 6" lifts in accordance Section 501.
2. AASHTO M 43, Size No. 57 stone shall be placed as shown behind the abutment and to the same geometrics behind the abutment wingwalls when they parallel the highway.
3. The Contractor must provide a well compacted surface to place the geotextile and stone fill against. In fill areas it will be necessary to overfill a sufficient distance so that a well compacted surface will be produced when the overfill is removed. The removed material will be paid for as Class 2 Excavation. The maximum pay limits for the stone will be the 2 ft. base and the slope as steep as ground will allow with the 1 : 2 maximum slope.
4. The cost of the stone and geotextile will be incidental to the pertinent structure concrete item.

FHWA APPROVAL
DATE:

APPROVAL
DEPUTY
CHIEF ENGR. BRIDGE DEVEL.
DATE:

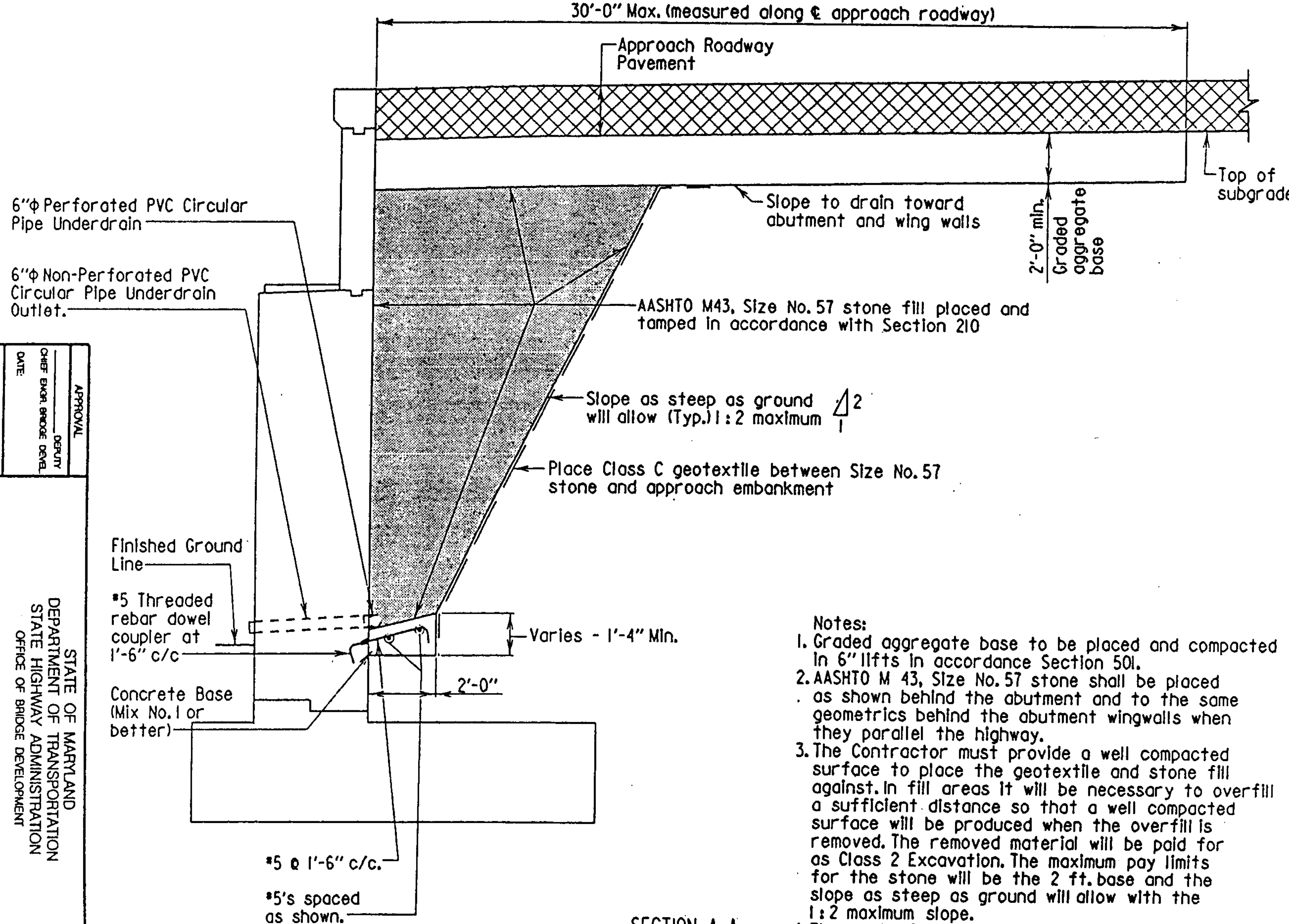
REVISIONS
SHA | FHWA

STATE OF MARYLAND
DEPARTMENT OF TRANSPORTATION
STATE HIGHWAY ADMINISTRATION
OFFICE OF BRIDGE DEVELOPMENT

DRAINAGE SYSTEM AND BACKFILL FOR ABUTMENTS (SEMI-CANTILEVER)

STANDARD NO. BR-SB(0.01)-80-101 SHEET 3 OF 4

SUBSTRUCTURE ABUT



SECTION A-A
Scale: None

Notes:

1. Graded aggregate base to be placed and compacted in 6" lifts in accordance Section 501.
2. AASHTO M 43, Size No. 57 stone shall be placed as shown behind the abutment and to the same geometrics behind the abutment wingwalls when they parallel the highway.
3. The Contractor must provide a well compacted surface to place the geotextile and stone fill against. In fill areas it will be necessary to overfill a sufficient distance so that a well compacted surface will be produced when the overfill is removed. The removed material will be paid for as Class 2 Excavation. The maximum pay limits for the stone will be the 2 ft. base and the slope as steep as ground will allow with the 1:2 maximum slope.
4. The cost of the stone and geotextile will be incidental to the pertinent structure concrete item.

FHWA APPROVAL
DATE:

APPROVAL
DEPUTY
CHIEF ENGR. BRIDGE DEVEL.
DATE:

REVISIONS
SHA FHWA

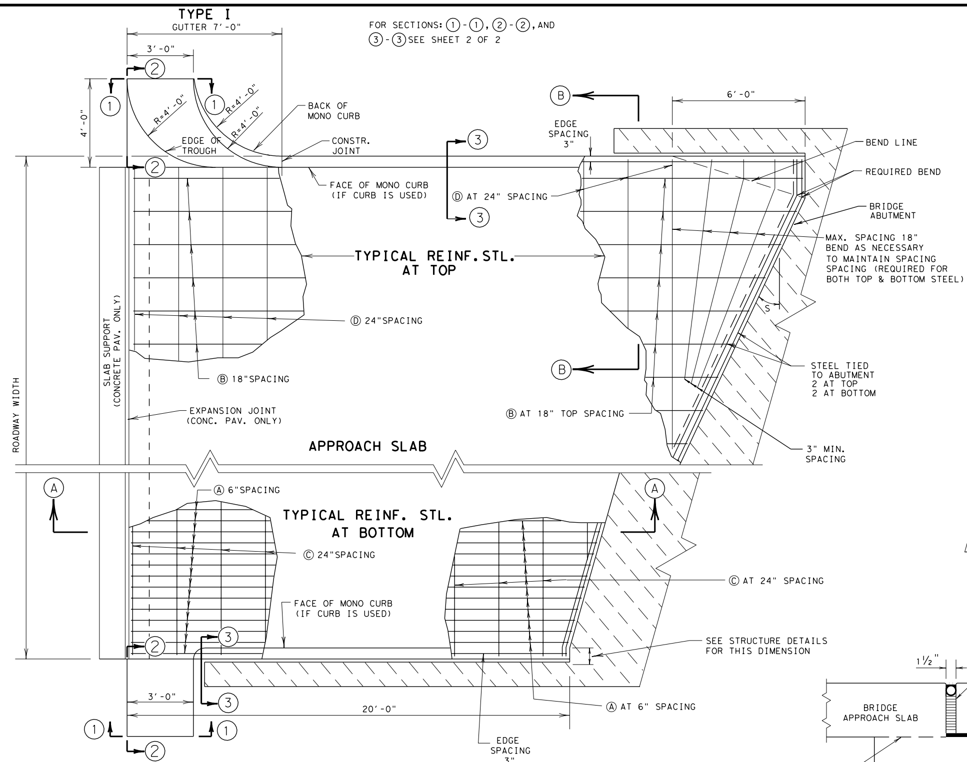
STATE OF MARYLAND
DEPARTMENT OF TRANSPORTATION
STATE HIGHWAY ADMINISTRATION
OFFICE OF BRIDGE DEVELOPMENT

DRAINAGE SYSTEM AND BACKFILL FOR ABUTMENTS (FULL CANTILEVER)

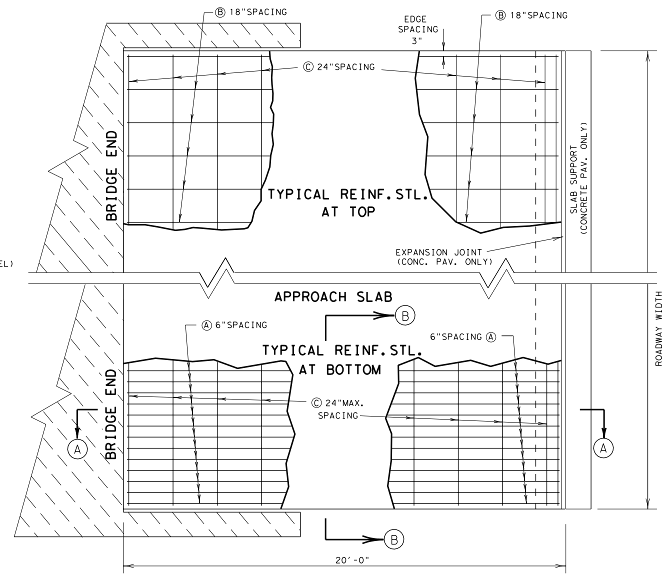
STANDARD NO. BR-SB(0.01)-80-101 SHEET 4 OF 4

SUBSTRUCTURE ABUT

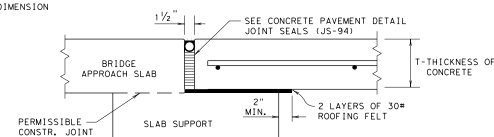
APPENDIX D APPROACH SLAB OF HOUSTON, Texas



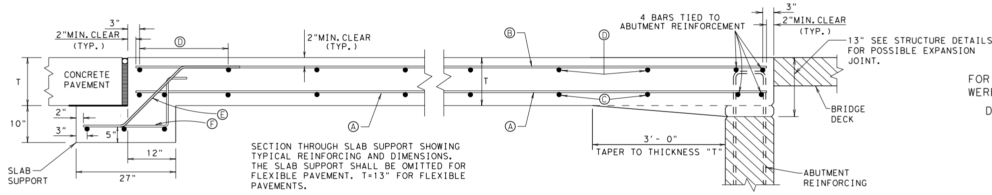
SKEWED BRIDGE APPROACH SLAB



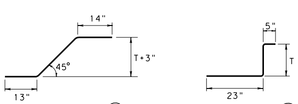
NORMAL BRIDGE APPROACH SLAB



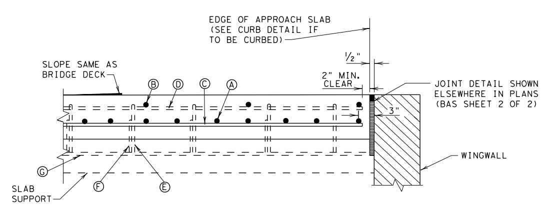
EXPANSION JOINT DETAIL A



TYPICAL SECTION A-A



BAR DETAIL E   BAR DETAIL F



TYPICAL SECTION B-B

FOR THE CONTRACTOR INFORMATION QUANTITIES WERE CALCULATED USING THE FOLLOWING :

DEFINITION OF TERMS

W = THE WIDTH OF THE SLAB IN FEET
S = ANGLE OF SKEW IN DEGREES
A_{BAS} = AREA OF BRIDGE APPROACH SLAB (S.Y.)

$$A_{BAS} = \frac{20W + \frac{W^2 \tan S}{2}}{9}$$

TABLE FOR STEEL QUANTITIES
STEEL POUNDS
38.0/S.Y. OF BAS
9.5/FT. OF SUPPORT SLAB

GENERAL NOTES

1. THE APPROACH SLAB SHALL HAVE THE SAME TRANSVERSE CROWN THROUGHOUT ITS LENGTH AS THE STRUCTURE WHICH IT ADJOINS.
2. ALL REINFORCEMENT BARS ON THIS STANDARD SHALL BE NUMBER 5 BARS. REINFORCING STEEL SHALL CONFORM TO ASTM DESIGNATION A-615, GRADE 60 OR A-616, GRADE 60. WHERE BENDING OF THE STEEL IS REQUIRED A-615, GRADE 60 SHALL BE USED.
3. ADDITIONAL DETAILS FOR POSSIBLE CURB AND GUTTER AND LONGITUDINAL JOINTS ARE SHOWN ON SHEET 2 OF 2 OF THIS STANDARD DRAWING.

Texas Department of Transportation
Design Division (Pavement)

BRIDGE APPROACH SLAB

FOR USE WITH ASPHALT AND CONCRETE PAVEMENTS NORMAL OR SKEW

BAS-94

SHEET 1 OF 2

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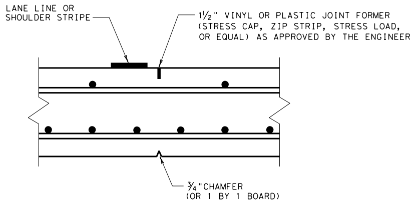
MODIFICATIONS | STATE DISTRICT | FEDERAL REGION 6 | FEDERAL AID PROJECT | SHEET | COUNTY | CONTROL | SECTION | JOB | HIGHWAY

165

LEVELS DISPLAYED | ACC: /usc/d481303 | FILE: BAS94.DGN

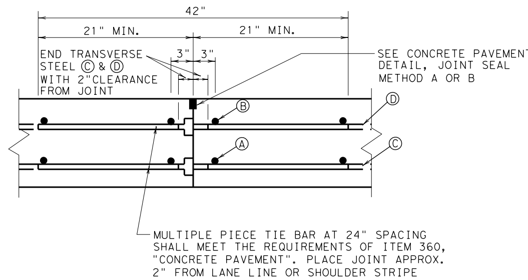
166



CONTROLLED LONGITUDINAL JOINT DETAIL

THE TOP JOINT MAY BE SAWED PROVIDED A LIGHT WEIGHT SAW IS USED WITHIN 6 HOURS OF PLACEMENT. A DRY SAWING OPERATION SHALL BE USED. THE SAW CUT SHALL BE LESS THAN 3/16 " WIDE.

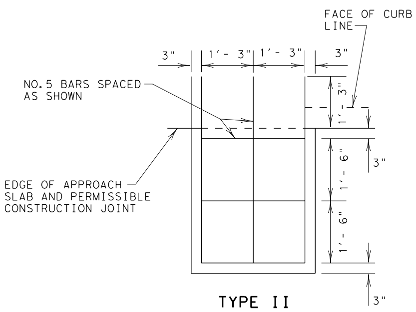
THE CHAMFER SHALL BE PLACED TO BE APPROXIMATELY MIDWAY BETWEEN THE LONGITUDINAL BOTTOM-STEEL. A REFERENCE LINE SHALL BE ESTABLISHED TO INSURE THAT THE INSERT OR SAW CUT IS DIRECTLY ABOVE THE CHAMFER. MINOR ADJUSTMENT IN THE A AND B BARS SPACINGS MAY BE REQUIRED TO INSURE THAT THE JOINT WILL BE APPROXIMATELY 2" FROM THE LANE LINE AND NOT ABOVE A REINFORCING BAR.



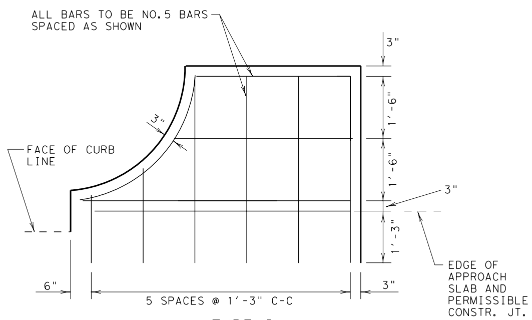
LONGITUDINAL CONSTRUCTION JOINT DETAIL

GENERAL NOTES

1. TIEBARS AT CONSTRUCTION JOINTS SHALL MEET THE REQUIREMENTS OF ITEM 360, "CONCRETE PAVEMENT."
2. IF A CURB IS USED ON THE STRUCTURE, A CURB WITH HEIGHT AND INSIDE FACE MATCHING THE STRUCTURE CURB SHALL BE USED ON THE APPROACH SLAB. THE CURB SHOULD TAPER TO A TWO INCH HEIGHT AND BE TERMINATED AT THE END OF THE SLAB OR AT THE DRAINAGE GUTTER, IF A DRAINAGE GUTTER IS USED.
3. DETAILS AND QUANTITIES FOR CURB ARE TO BE AS SHOWN ELSEWHERE IN THE PLANS.
4. TYPE I GUTTER SHOULD BE USED IN LIEU OF TYPE II GUTTER WHEN SPACE PERMITS.
5. WHEN STAGE CONSTRUCTION IS USED TO BUILD A PORTION OF A BRIDGE, THE LONGITUDINAL CONSTRUCTION JOINTS OF THE APPROACH SLAB SHALL ALIGN WITH THE LONGITUDINAL CONSTRUCTION JOINTS OF OF THE BRIDGE STRUCTURE.
6. ALL REINFORCEMENT STEEL ON THIS STANDARD SHEET SHALL BE NUMBER 5 (#5)BARS.

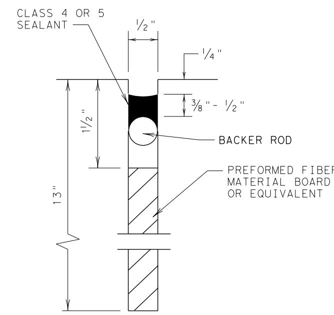


TYPE II



TYPE I

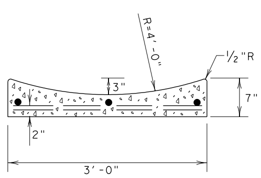
REINFORCING LAYOUT FOR GUTTER



EXPANSION JOINT

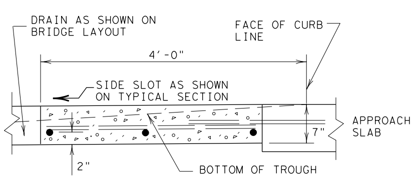
BETWEEN SLAB AND WINGWALL

PLACE IN ACCORDANCE WITH ITEM 438 "CLEANING AND/OR SEALING JOINTS AND CRACKS".

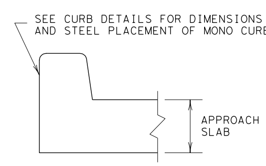


SECTION AT ① - ①

TYPES I & II GUTTER



SECTION AT ② - ②



SECTION AT ③ - ③

SHOWING MONO CURB

Texas Department of Transportation

Design Division (Pavement)

BRIDGE APPROACH SLAB

FOR USE WITH ASPHALT AND CONCRETE PAVEMENTS NORMAL OR SKEW

BAS-94

SHEET 2 OF 2

©TxDOT SEPTEMBER 1994	DN: LJB	CK: LJB	DW: BGD	CK: GLG	NEG NO.: R0000
MODIFICATIONS	STATE DISTRICT	FEDERAL REGION	FEDERAL AID PROJECT	*	SHEET
		6			
	COUNTY		CONTROL	SECTION	JOB	HIGHWAY

ACC: /usc/d481303

FILE: BAS94.DGN

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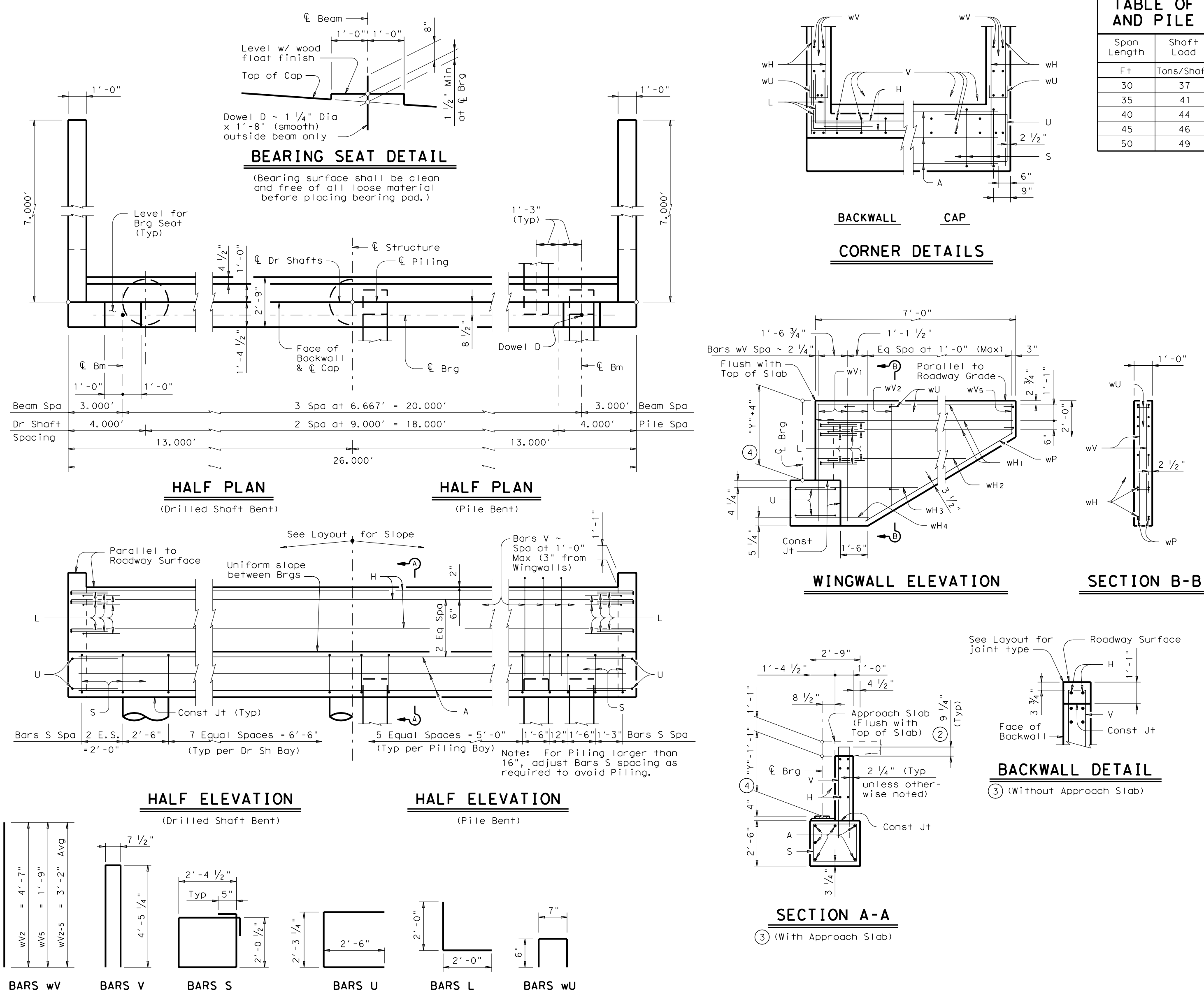


TABLE OF SHAFT AND PILE LOADS

Span Length	Shaft Load	Pile Load
Ft	Tons/Shaft	Tons/Pile
30	37	31
35	41	32
40	44	34
45	46	35
50	49	37

(1) TABLE OF ESTIMATED QUANTITIES

Bar	No.	Size	Length	Weight
A	6	#11	25'- 0"	797
D	2	1 1/4"D	1'- 8"	14
H	6	#5	25'- 8"	161
L	18	#6	4'- 0"	108
S	22	#4	9'- 8"	142
U	4	#6	7'- 3"	44
V	25	#5	9'- 6"	248
wH 1	12	#6	6'- 8"	120
wH 2	4	#6	5'- 6"	33
wH 3	4	#6	5'- 9"	35
wH 4	4	#6	4'- 0"	24
wP	4	#6	5'- 8"	34
wU	6	#4	1'- 7"	6
wV 1	12	#5	5'- 7"	70
wV 2-5	16	#5	3'- 2"Av	53
Reinforcing Steel			Lb	1889
Class "C" Concrete			CY	11.1

(1) Quantities shown are for one Abutment only (with Approach Slab). With no Approach Slab, add 1.0 CY Class "C" Concrete and 54 Lb Reinforcing Steel for 2 additional H Bars.

(2) Increase as required to maintain 3 3/4" from Finished Grade.

(3) See Layout to determine if Approach Slab is present.

(4) See Span sheet for "Y".

GENERAL NOTES:

Designed according to current AASHTO Standard and Interim Specifications.
Concrete strength f'c = 3,600 psi.
All cap and wall reinforcing shall be Grade 60.
Designed for normal embankment header slope of 2 to 1 and a maximum span length of 50 Ft.
See Layout for foundation type.
See Foundation Detail Standard Sheet, FD, for all foundation details and notes.

HS20 LOADING

Texas Department of Transportation
Design Division (Bridge)

ABUTMENTS
TY A PRESTR CONC I-BMS 24' ROADWAY 2:1 HEADER SLOPE

AIB2-A-24

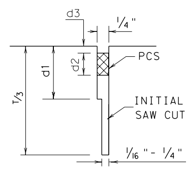
FILE: abste001.dgn	DN: TxDOT	CK: TxDOT	DW: JRP	CK: LDS
©TxDOT April 1999	DISTRICT	FEDERAL AID PROJECT		SHEET
REVISIONS	COUNTY	CONTROL	SECT JOB	HIGHWAY

LEVELS DISPLAYED ACC:

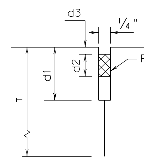
168

ACC: /usc/d481303

FILE: JS94.DGN

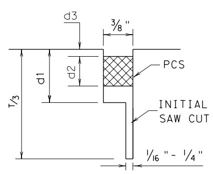


SAWED
LONGITUDINAL JOINT

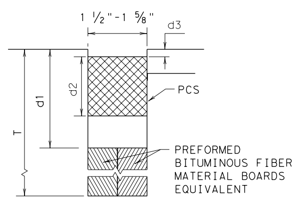


LONGITUDINAL
CONSTRUCTION JOINT

LONGITUDINAL JOINT SEALS



SAWED
CONTRACTION JOINT

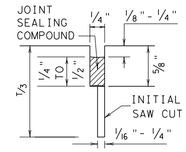


FORMED
FORMED EXPANSION JOINT

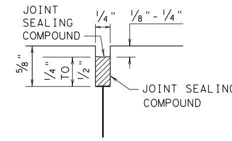
TRANSVERSE JOINT SEALS

METHOD A: PREFORMED COMPRESSION SEALS (PCS)

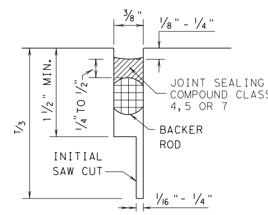
(CLASS 6 PREFORMED JOINT SEALANT)



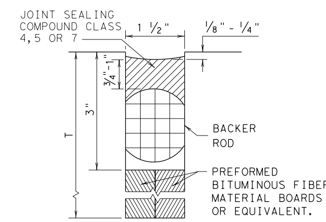
SAWED
LONGITUDINAL JOINT



LONGITUDINAL OR TRANSVERSE
CONSTRUCTION JOINT



TRANSVERSE SAWED
CONTRACTION JOINT



TRANSVERSE FORMED
EXPANSION JOINT

METHOD B: JOINT SEALING COMPOUND

GENERAL NOTES FOR METHOD "A"

1. UNLESS OTHERWISE SHOWN IN THE PLANS, EITHER METHOD "A" OR METHOD "B" MAY BE USED.
2. THE LOCATION OF JOINTS SHALL BE AS SHOWN ELSEWHERE IN THE PLANS.
3. DIMENSIONS d1, d2, AND d3 SHALL BE IN ACCORDANCE WITH THE PREFORMED COMPRESSION SEAL MANUFACTURES RECOMMENDATION.
4. THE JOINT RESERVOIR FOR SEALANT SHALL BE SAWED UNLESS OTHERWISE SHOWN ON THE PLANS FOR THE LONGITUDINAL AND TRANSVERSE CONSTRUCTION AND THE TWO SAWED JOINTS.
5. THE JOINTS SHALL BE CLEANED IN ACCORDANCE WITH THE ITEM 438 AND PRIOR TO BEGINNING OPERATIONS, THE CONTRACTOR SHALL SUBMIT A STATEMENT FROM THE SEALANT MANUFACTURER SHOWING THE RECOMMENDED EQUIPMENT AND INSTALLATION PROCEDURES TO BE USED.
6. THE SAW CUT FOR THE LONGITUDINAL JOINT SHALL BE ONE FOURTH THE SLAB THICKNESS WHEN CRUSHED LIMESTONE IS USED AS THE COARSE AGGREGATE.

GENERAL NOTES FOR METHOD "B"

1. UNLESS OTHERWISE SHOWN IN THE PLANS, EITHER METHOD "A" OR METHOD "B" MAY BE USED.
2. THE LOCATION OF JOINTS SHALL BE AS SHOWN ELSEWHERE IN THE PLANS.
3. THE ENGINEER SHALL SELECT A TARGET PLACEMENT THICKNESS FOR THE SEALANT DETAILS WHICH SHOW RANGES IN THICKNESS. THE TARGET THICKNESS WILL NORMALLY BE THE MIDPOINT OF THE RANGE.
4. THE JOINT RESERVOIR FOR SEALANT SHALL BE SAWED UNLESS OTHERWISE SHOWN ON THE PLANS FOR THE LONGITUDINAL AND TRANSVERSE CONSTRUCTION AND THE TWO SAWED JOINTS.
5. THE JOINTS SHALL BE CLEANED IN ACCORDANCE WITH THE ITEM 438 AND PRIOR TO BEGINNING OPERATIONS, THE CONTRACTOR SHALL SUBMIT A STATEMENT FROM THE SEALANT MANUFACTURER SHOWING THE RECOMMENDED EQUIPMENT AND INSTALLATION PROCEDURES TO BE USED.
6. THE SAW CUT FOR THE LONGITUDINAL JOINT SHALL BE ONE FOURTH THE SLAB THICKNESS WHEN CRUSHED LIMESTONE IS USED AS THE COARSE AGGREGATE.

Texas Department of Transportation
Design Division (Pavement)

CONCRETE PAVING DETAILS
JOINT SEALS

JS-94

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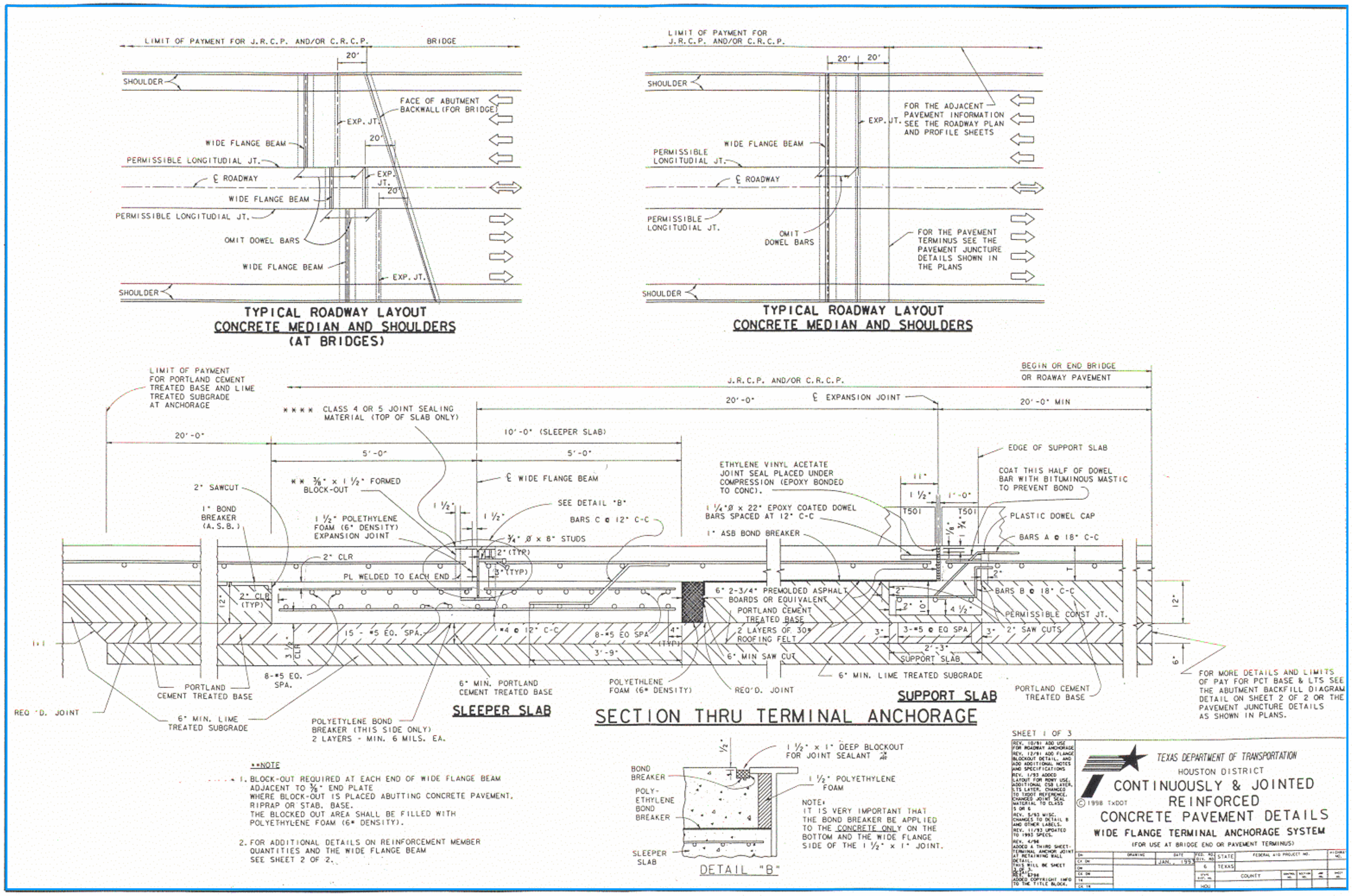
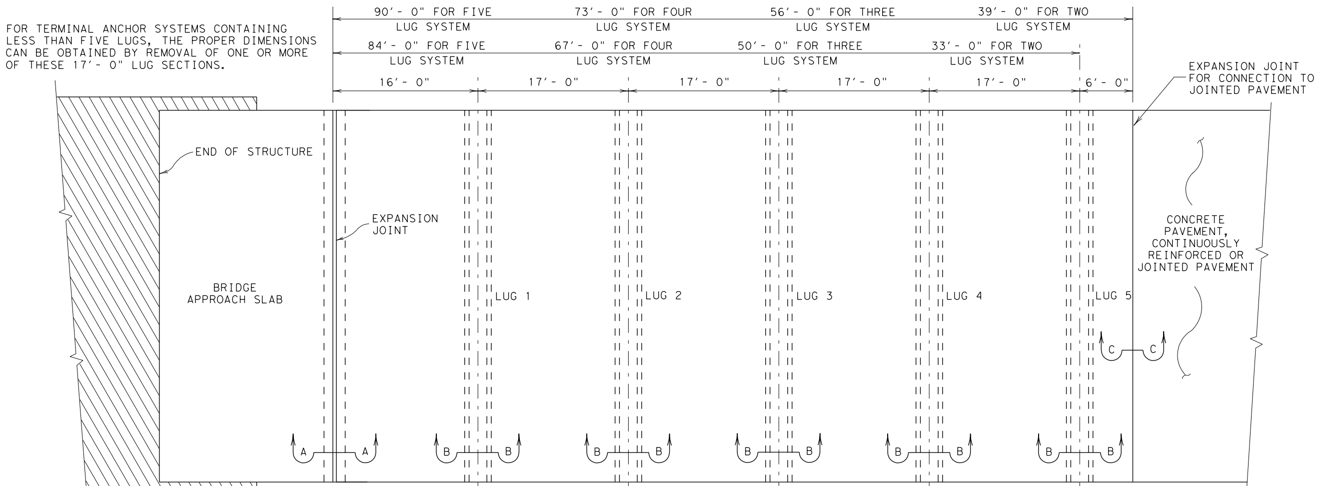
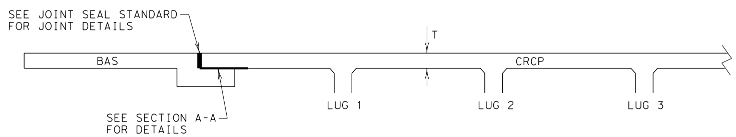
MODIFICATIONS	STATE DISTRICT	FEDERAL REGION	FEDERAL AID PROJECT		SHEET
		6			
	COUNTY		CONTROL	SECTION	JOB	HIGHWAY

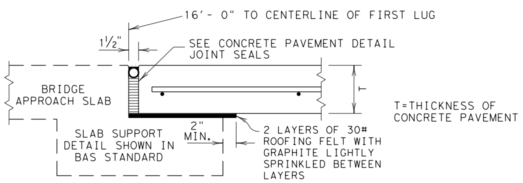


LIMIT OF PAYMENT FOR J.R.C.P. AND/OR C.R.C.P.
BRIDGE
SHOULDER
FACE OF ABUTMENT BACKWALL (FOR BRIDGE)
EXP. JT.
WIDE FLANGE BEAM
PERMISSIBLE LONGITUDIAL JT.
℄ ROADWAY
OMIT DOWEL BARS
TYPICAL ROADWAY LAYOUT
CONCRETE MEDIAN AND SHOULDERS
(AT BRIDGES)
LIMIT OF PAYMENT FOR J.R.C.P. AND/OR C.R.C.P.
FOR THE ADJACENT PAVEMENT INFORMATION SEE THE ROADWAY PLAN AND PROFILE SHEETS
FOR THE PAVEMENT TERMINUS SEE THE PAVEMENT JUNCTURE DETAILS SHOWN IN THE PLANS
TYPICAL ROADWAY LAYOUT
CONCRETE MEDIAN AND SHOULDERS
LIMIT OF PAYMENT FOR PORTLAND CEMENT TREATED BASE AND LIME TREATED SUBGRADE AT ANCHORAGE
J.R.C.P. AND/OR C.R.C.P.
BEGIN OR END BRIDGE OR ROAWAY PAVEMENT
℄ EXPANSION JOINT
20'-0" MIN
**** CLASS 4 OR 5 JOINT SEALING MATERIAL (TOP OF SLAB ONLY)
10'-0" (SLEEPER SLAB)
EDGE OF SUPPORT SLAB
** 3/8" x 1 1/2" FORMED BLOCK-OUT
℄ WIDE FLANGE BEAM
ETHYLENE VINYL ACETATE JOINT SEAL PLACED UNDER COMPRESSION (EPOXY BONDED TO CONC).
COAT THIS HALF OF DOWEL BAR WITH BITUMINOUS MASTIC TO PREVENT BOND
2" SAWCUT
1" BOND BREAKER (A.S.B.)
1 1/2" POLETHYLENE FOAM (6# DENSITY) EXPANSION JOINT
SEE DETAIL "B"
BARS C @ 12" C-C
1 1/4"Ø x 22" EPOXY COATED DOWEL BARS SPACED AT 12" C-C
PLASTIC DOWEL CAP
1" ASB BOND BREAKER
BARS A @ 18" C-C
3/4" Ø x 8" STUDS
2" CLR
PL WELDED TO EACH END
6" 2-3/4" PREMOLDED ASPHALT BOARDS OR EQUIVALENT
BARS B @ 18" C-C
PORTLAND CEMENT TREATED BASE
PERMISSIBLE CONST JT.
2 LAYERS OF 30# ROOFING FELT
2" SAW CUTS
15 - #5 EQ. SPA.
#4 @ 12" C-C
8-#5 EQ SPA
3-#5 @ EQ SPA
SUPPORT SLAB
6" MIN SAW CUT
6" MIN. LIME TREATED SUBGRADE
8-#5 EQ. SPA.
PORTLAND CEMENT TREATED BASE
6" MIN. PORTLAND CEMENT TREATED BASE
POLYETHLENE FOAM (6# DENSITY)
REQ'D. JOINT
SUPPORT SLAB
PORTLAND CEMENT TREATED BASE
FOR MORE DETAILS AND LIMITS OF PAY FOR PCT BASE & LTS SEE THE ABUTMENT BACKFILL DIAGRAM DETAIL ON SHEET 2 OF 2 OR THE PAVEMENT JUNCTURE DETAILS AS SHOWN IN PLANS.
REQ'D. JOINT
6" MIN. LIME TREATED SUBGRADE
POLYETYLENE BOND BREAKER (THIS SIDE ONLY) 2 LAYERS - MIN. 6 MILS. EA.
SLEEPER SLAB
SECTION THRU TERMINAL ANCHORAGE
**NOTE
1. BLOCK-OUT REQUIRED AT EACH END OF WIDE FLANGE BEAM ADJACENT TO 3/8" END PLATE WHERE BLOCK-OUT IS PLACED ABUTTING CONCRETE PAVEMENT, RIPRAP OR STAB. BASE. THE BLOCKED OUT AREA SHALL BE FILLED WITH POLYETHYLENE FOAM (6# DENSITY).
2. FOR ADDITIONAL DETAILS ON REINFORCEMENT MEMBER QUANTITIES AND THE WIDE FLANGE BEAM SEE SHEET 2 OF 2.
1 1/2" x 1" DEEP BLOCKOUT FOR JOINT SEALANT
BOND BREAKER
1 1/2" POLYETHYLENE FOAM
POLY-ETHYLENE BOND BREAKER
NOTE: IT IS VERY IMPORTANT THAT THE BOND BREAKER BE APPLIED TO THE CONCRETE ONLY ON THE BOTTOM AND THE WIDE FLANGE SIDE OF THE 1 1/2" x 1" JOINT.
SLEEPER SLAB
DETAIL "B"
SHEET 1 OF 3
TEXAS DEPARTMENT OF TRANSPORTATION
HOUSTON DISTRICT
© 1998 TxDOT
CONTINUOUSLY & JOINTED REINFORCED CONCRETE PAVEMENT DETAILS
WIDE FLANGE TERMINAL ANCHORAGE SYSTEM
(FOR USE AT BRIDGE END OR PAVEMENT TERMINUS)
JAN., 1993
TEXAS
HOU

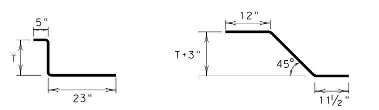
FOR TERMINAL ANCHOR SYSTEMS CONTAINING LESS THAN FIVE LUGS, THE PROPER DIMENSIONS CAN BE OBTAINED BY REMOVAL OF ONE OR MORE OF THESE 17'- 0" LUG SECTIONS.





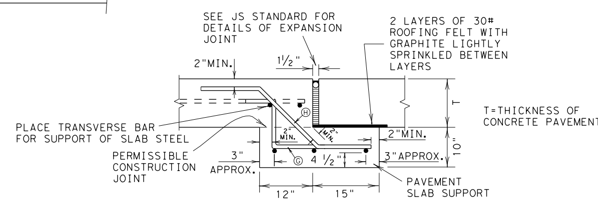


EXPANSION JOINT
TYPICAL SECTION A-A



(G) #5 BAR AT 18" (H) #5 BAR AT 18"

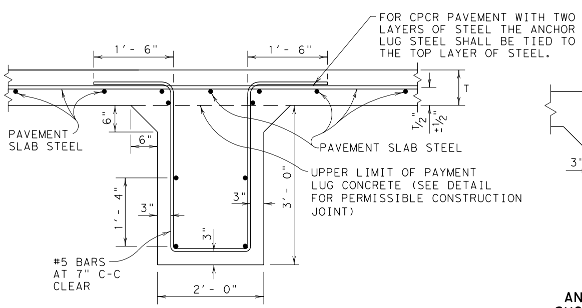
BAR SPACING SHALL AVERAGE 18" SPACING MAY VARY ½ 2" TO FIT BETWEEN PAVEMENT STEEL



TYPICAL SECTION C-C
EXPANSION JOINT FOR JOINTED CONCETE PAVEMENT TERMINAL

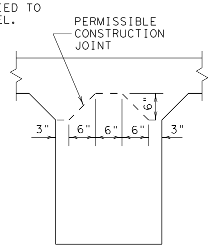
GENERAL NOTES

1. THE BRIDGE APPROACH SLAB DETAILS ARE AS SHOWN ELSEWHERE IN THE PLANS.
2. THE LOCATIONS OF THE TERMINAL ANCHORAGE SYSTEMS AND NUMBER OF LUGS PER SYSTEM SHALL BE AS SHOWN ELSEWHERE IN THE PLANS.
3. DETAILS AS TO PAVEMENT SLAB WIDTH, THICKNESS, AND CROWN CROSS-SLOPE SHALL BE AS SHOWN ELSEWHERE ON THE PLANS.
4. THE CONCRETE PAVEMENT SLAB SHALL CONTINUE ACROSS THE ANCHOR LUGS AS SHOWN. ALL REQUIREMENTS AS TO CONSTRUCTION MEASUREMENT, AND PAYMENT FOR THE PAVEMENT SLAB SHALL BE MADE UNDER THE ITEM "CONCRETE PAVEMENT." DETAILS FOR THE SLAB SHALL BE SHOWN IN THE "CONCRETE PAVEMENTS DETAILS, CONTINUOUSLY REINFORCED STEEL BARS.
5. REINFORCING FOR THE ANCHOR LUGS SHALL BE STEEL CONFORMING TO ASTM DESIGNATION A615, GRADE 60 OR A-616, GRADE 60. BARS TO BE BEND SHALL BE A -615, GRADE 60. ALL STEEL IN LUGS AND SUPPORT SLAB SHALL BE #5 BARS.
6. DETAILS OF LONGITUDINAL JOINTS (TRANSVERSE JOINTS, IF NECESSARY) AND REQUIRED JOINT SEALS ARE SHOWN ON THE APPROPRIATE PAVING DETAIL SHEET.
7. TRANSVERSE CONSTRUCTION JOINTS WILL NOT BE PERMITTED WITHIN THE LIMITS OF THE ANCHOR LUGS WITHOUT THE APPROVAL OF THE ENGINEER.



ANCHOR LUG DETAIL

SECTION B-B



ANCHOR LUG DETAIL SHOWING PERMISSIBLE CONSTRUCTION JOINT

MATERIAL	ESTIMATING QUANTITIES FOR ANCHOR LUG	
	24 FOOT LENGTH	PER FOOT OF LENGTH
EXCAVATION AND CONCRETE (CU. YD.)	6.2	0.26
*STEEL (LBS.)	631	26.3

QUANTITIES FOR SUPPORT SLAB USED WITH JOINTED CONCRETE (CONCRETE, EXCAVATION AND STEEL PART OF CONCRETE PAVEMENT ITEM		
MATERIAL	24 FOOT LENGTH	PER FOOT OF LENGTH
EXCAVATION AND CONCRETE (CU. YD.)	1.7	0.07
*STEEL (LBS.)	144	6

*STEEL WEIGHT WILL INCREASE SLIGHTLY FOR PAVEMENTS THICKER THAN 10".

Texas Department of Transportation
Design Division (Pavement)

TERMINAL ANCHORAGE FOR CONCRETE PAVEMENT

TA(CP)-99

FILE: TACP99.DGN	DN: LJB	CK: LJB	DW: BGD	CK:	
© TxDOT September 1994	DIST	FED REG	FEDERAL AID PROJECT		SHEET
REVISIONS		6			
	COUNTY		CONTROL	SECT	JOB	HIGHWAY

170

LEVELS DISPLAYED

APPENDIX E COMPACTION SPECIFICATIONS OF HOUSTON, Texas

When proof rolling is shown on the plans and directed by the Engineer, it will be paid for in accordance with Item 216, "Rolling (Proof)".

ITEM 132

EMBANKMENT

132.1. Description. This Item shall govern for the placement and compaction of all materials necessary for the construction of roadway embankments, levees and dykes or any designated section of the roadway where additional material is required.

132.2. Material. Materials may be furnished from required excavation in the areas shown in the plans or from off right of way sources obtained by the Contractor and meeting the requirements herein. All embankment shall conform to one of the following types as shown on the plans, except that material which is in a retaining-wall-backfill area shall meet the requirements for backfill material of the pertinent retaining-wall item:

Type A. This material shall consist of suitable granular material, free from vegetation or other objectionable matter, and reasonably free from lumps of earth. This material shall be suitable for forming a stable embankment and, when tested in accordance with Test Methods Tex-104-E, Tex-105-E, Tex-106-E and Tex-107-E, Part II shall meet the following requirements:

The liquid limit shall not exceed	45
The plasticity index shall not exceed.	15
The bar linear shrinkage shall not be less than	2

Type B. This material shall consist of suitable earth material such as rock, loam, clay, or other such materials as approved by the Engineer that will form a stable embankment.

Type C. This material shall be suitable and shall conform to the specification requirements shown on the plans.

Type D. This material shall be that obtained from required excavation areas shown on the plans and will be used in embankment.

132.3. Construction Methods.

(1) General. When off right of way sources are involved, the Contractor's attention is directed to Item 7, "Legal Relations and Responsibilities to the Public". Prior to placing any embankment, all work in accordance with Item 100, "Preparing Right of Way", shall have been completed on the areas over which the embankment is to be placed. Stump holes or other small excavations in the limits of the embankments shall be backfilled with suitable material and thoroughly tamped by approved methods before commencing embankment construction. The surface of the ground, including disk-loosened ground or any surface roughened by small washes or otherwise, shall be restored to approximately its original slope by blading or other methods. Where shown on the plans or required by the Engineer, the ground surface thus prepared shall be compacted by sprinkling and rolling.

The Engineer shall be notified sufficiently in advance of opening any material source to allow performance of any required testing.

Unless otherwise shown on the plans, the surfaces of unpaved areas (except rock) which are to receive embankment shall be loosened by scarifying to a depth of at least 150 millimeters. Hillsides shall be cut into steps before embankment materials are placed. Placement of embankment materials shall begin at the low side of hillsides and slopes. Materials which have been loosened shall be recompacted simultaneously with the new embankment materials placed upon it. The total depth of loosened and new materials shall not exceed the permissible depth of the layer to be compacted, as specified in Subarticle 132.3.(3).(a) and (b).

Trees, stumps, roots, vegetation or other unsuitable materials shall not be placed in embankment.

Unless otherwise shown on the plans, all embankment shall be constructed in layers approximately parallel to the finished grade of the roadbed.

Embankments shall be constructed to the grade and sections shown on the plans or as established by the Engineer. Each section of the embankment shall correspond to the detailed section or slopes established by the Engineer. After completion of the roadway, it shall be continuously maintained to its finished section and grade until the project is accepted.

(2) Constructing Embankments.

(a) **Earth Embankments.** Earth embankments shall be defined as those composed principally of material other than rock, and shall be constructed of acceptable material from approved sources.

Unless otherwise specified, earth embankments shall be constructed in successive layers for the full width of the individual roadway cross section and in such lengths as are best suited to the sprinkling and compacting methods utilized.

Layers of embankment may be formed by utilizing equipment and methods which will evenly distribute the material.

A minor quantity of rock or broken concrete encountered in the construction of this project may be incorporated in the lower layers of the embankment if acceptable to the Engineer. Or, it may be placed in the deeper fills, in accordance with the requirements for the construction of rock embankments, provided such placement of rock is not immediately adjacent to structures or in areas where bridge foundations are to be constructed. Also, rock or broken concrete may be placed in the portions of embankments outside the limits of the completed roadbed width where the size of the rock or broken concrete prohibits its incorporation in the normal embankment layers. All exposed reinforced steel shall be cut and removed from the broken concrete.

Each layer of embankment shall be uniform as to material, density and moisture content before beginning compaction. Where layers of unlike materials abut each other, each layer shall be featheredged for at least 30 meters, or the material shall be so mixed as to prevent abrupt changes in the soil. No material placed in the embankment by dumping in a pile or windrow shall be incorporated in a layer in that position, but all such piles or windrows shall be moved by blading or similar methods. Clods or lumps of material shall be broken and the embankment material mixed by blading, harrowing, disking or similar methods until a uniform material of uniform density is achieved in each layer.

Sprinkling required to achieve the moisture content necessary for compaction shall meet the material requirements of Item 204, "Sprinkling". It shall be the responsibility of the Contractor to secure a uniform moisture content throughout the layer by such methods as may be necessary. In order to facilitate uniform wetting of the embankment material, the Contractor may apply water at the material source if the sequence and

methods used do not cause an undue waste of water. Such procedures shall be subject to the approval of the Engineer.

(b) **Rock Embankments.** Rock embankments shall be defined as those composed principally of rock, and shall be constructed of acceptable material.

Unless otherwise specified, rock embankments normally shall be constructed in successive layers for the full width of the individual roadway cross section and of 450 millimeters or less in depth. When, in the opinion of the Engineer, the rock sizes necessitate a greater depth of layer, the layer depth may be increased as necessary, but in no case shall the depth of layer exceed 0.75 meter. Each layer shall be constructed in such a manner that the interstices between the larger stones are filled with smaller stones and spalls which have been created by this operation as well as from the placement of succeeding layers of material.

The maximum dimension of any rock used in embankment shall be less than the depth of the embankment layer, and in no case shall any rock over 0.6 meter in its greatest dimension be placed in the embankment unless otherwise approved by the Engineer. Unless otherwise shown on the plans, the upper or final layer of the embankment shall be composed of material so graded that the density and uniformity of the surface layer may be secured by the "Ordinary Compaction" or "Density Control" method. Exposed oversize material shall be reduced by sledging or other methods as approved by the Engineer.

When "Ordinary Compaction" is specified, each embankment layer shall be rolled and sprinkled when and to the extent directed by the Engineer. When "Density Control" is specified, each layer shall be compacted to the required density as outlined for "Earth Embankments", except that in those layers where rock will make density testing difficult, when shown on the plans, the Engineer may require the layer to be proof rolled to insure proper compaction.

(c) **Embankment Adjacent to Culverts and Bridges.** Embankments adjacent to culverts and bridges shall be compacted in the manner prescribed under Item 400, "Excavation and Backfill for Structures", or other appropriate bid items.

As a general practice, embankment material placed adjacent to any portion of any structure and in the first two layers above the top of any culvert or similar structure shall be free of any appreciable amount of gravel or stone particles more than 100 millimeters in greatest dimension and of such gradation as to permit thorough compaction. When, in the opinion of the Engineer, such material is not readily available, the use of rock or gravel mixed with earth will be permitted, in which case no particle larger than 300 millimeters in greatest dimension and 150 millimeters in least dimension may be used. The percentage of fines shall be sufficient to fill all voids and insure a uniform and thoroughly compacted mass of proper density.

(3) **Compaction Methods.** Compaction of embankments shall be by "Ordinary Compaction" or "Density Control" as shown on the plans.

(a) **Ordinary Compaction.** When "Ordinary Compaction" is shown on the plans, the following provisions shall govern:

Each layer shall not exceed 200 millimeters of loose depth, unless otherwise directed by the Engineer. Each layer shall be compacted in accordance with the provisions governing the Item or Items of "Rolling". Unless otherwise specified on the plans, the rolling equipment shall be as approved by the Engineer. Compaction shall continue until there is no evidence of further compaction. Prior to and in conjunction with the rolling operation, each layer shall be brought to the moisture content directed by the Engineer, and shall be kept leveled with suitable equipment to insure uniform compaction over the entire layer. Should the subgrade, for any reason or cause, lose the required stability or finish, it shall be recompacted and refinished at the Contractor's expense.

(b) **Density Control.** When "Density Control" is shown on the plans, the following provisions shall apply:

Each layer shall be compacted to the required density by any method, type and size of equipment which will give the required compaction. The depth of layers, prior to compaction, shall depend upon the type of sprinkling, mixing and compacting equipment used. However, maximum depth (400 millimeters loose and 300 millimeters compacted) shall not be exceeded unless approved by the Engineer. Prior to and in conjunction with the rolling operation, each layer shall be brought to the moisture content necessary to obtain the required density and shall be kept leveled with suitable equipment to insure uniform compaction over the entire layer.

Each layer shall be sprinkled as required and compacted to the extent necessary to provide the density specified below, unless otherwise shown on the plans.

Description	Density, Percent	Moisture
Non-swelling soils with plasticity index less than 20	Not less than 98	
Swelling soils with plasticity index of 20 to 35	Not less than 98 nor more than 102	Not less than optimum
Swelling soils with plasticity index over 35	Not less than 95 nor more than 100	Not less than optimum

The density determination will be made in accordance with Test Method Tex-114-E. Field density determination will be made in accordance with Test Method Tex-115-E.

After each layer of earth embankment is complete, tests as necessary may be made by the Engineer. When the material fails to meet the density requirements or should the material lose the required stability, density, moisture or finish before the next course is placed or the project is accepted, the layer shall be reworked as necessary to obtain the specified compaction, and the compaction method shall be altered on subsequent work to obtain specified density. Such procedure shall be subject to the approval of the Engineer.

Excessive loss of moisture shall be construed to exist when the subgrade soil moisture content is four (4) percent less than the optimum.

The Contractor may be required to remove a small area of the layer in order to facilitate the taking of density tests. Replacement and compaction of the removed material in the small area shall be at the Contractor's expense.

When shown on the plans and when directed by the Engineer, the Contractor shall proof roll in accordance with Item 216, "Rolling (Proof)". Soft spots shall be corrected as directed by the Engineer.

132.4. **Tolerances.** The tolerances shall be as follows:

(1) **Grade Tolerances.**

(a) **Stage Construction.** Any deviation in excess of 30 millimeters in cross section and 30 millimeters in five (5) meters measured longitudinally shall be corrected by loosening, adding or removing the material, reshaping and recompacting by sprinkling and rolling.

(b) **Turnkey Construction.** Any deviation in excess of 15 millimeters in cross section and 15 millimeters in five (5) meters measured longitudinally shall be corrected by loosening, adding or removing the material, reshaping and recompacting by sprinkling and rolling.

(2) **Gradation Tolerances.** The Engineer may accept the material, providing not more than one (1) out of the most recent five (5) gradation tests performed are outside the specified limit on any individual sieve by more than five (5) percent.

(3) **Density Tolerances.** The Engineer may accept the work providing not more than one (1) out of the most recent five (5) density tests performed is outside the specified density, provided the failing test is no more than 50 kilograms per cubic meter outside the specified density.

(4) **Plasticity Tolerances.** The Engineer may accept the material providing not more than one (1) out of the most recent five (5) plasticity index samples tested are outside the specified limit by no more than two (2) points.

132.5. **Measurement.** This Item will be measured as follows:

(1) **General.**

Retaining-wall-backfill areas which are also in embankment areas will be measured for payment as embankment except as shown on the plans; such material shall meet the requirements for backfill material of the pertinent retaining-wall item(s). Limits of measurement for embankment in retaining-wall areas will be as shown on Standard Detail Sheet "Earthwork Measurement at Retaining Walls" (EMRW) in the plans.

Shrinkage or swellage factors will not be considered in determining the calculated quantities.

(2) **Class 1.** Embankment will be measured in its original, natural position, and the volume computed in cubic meters by the method of average end area.

(3) **Class 2.** Embankment will be measured by the cubic meter in vehicles as delivered on the road.

(4) **Class 3.** Embankment will be measured by the cubic meter in its final position as the volume of embankment computed in place between (1) the original ground surfaces or the surface upon which the embankment is to be constructed, and (2) the lines, grades and slopes of the accepted embankment, using the average end area method.

Class 3 is a plans quantity measurement Item and the quantity to be paid for will be that quantity shown in the proposal and on the "Estimate and Quantity" sheet of the contract plans, except as may be modified by Article 9.8. If no adjustment of quantities is required, additional measurements or calculations will not be required.

132.6. **Payment.** The work performed and materials furnished in accordance with this Item and measured as provided under "Measurement" will be paid for at the unit price bid for "Embankment", of the compaction method, type and class specified. This price shall be full compensation for furnishing embankment; for hauling; for placing, compacting, finishing and reworking; and for all labor, royalty, tools, equipment and incidentals necessary to complete the work.

When proof rolling is shown on the plans and directed by the Engineer, it will be paid for in accordance with Item 216, "Rolling (Proof)".

When "Ordinary Compaction" is shown on the plans, all sprinkling and rolling, except proof rolling, will not be paid for directly, but will be considered subsidiary to this Item, unless otherwise shown on the plans.

When "Density Control" is shown on the plans, all sprinkling and rolling, except proof rolling, will not be paid for directly, but will be considered subsidiary to this Item.

When subgrade is constructed under this project, correction of soft spots in the subgrade will be at the Contractor's expense. When subgrade is not constructed under this project, correction of soft spots in the subgrade will be in accordance with Article 4.3.

ITEM 134

BACKFILLING PAVEMENT EDGES

134.1. Description. This Item shall govern for backfilling pavement edges in conformity with widths and typical sections shown on the plans. This Item also includes the application of an emulsified asphalt and/or fertilizer with the backfill material, when specified on the plans.

134.2. Material.

(1) **General.** Unless otherwise indicated on the plans, the top 100 millimeters of the backfill material shall be capable of sustaining vegetation. When less than 100 millimeters of backfill is required, the material supplied shall be capable of sustaining vegetative growth.

(2) **Backfill Material.** Backfill material shall be one of the following types:

Type A. Backfill material shall be provided from a source outside the right of way and be in accordance with the requirements shown on the plans.

Type B. Backfill material shall be secured from within the existing right of way as shown on the plans or as directed by the Engineer.

Type C. Backfill material shall be mulch sodding provided from an approved source in accordance with Subarticle 162.3(8).

(3) **Emulsified Asphalt.** The emulsified asphalt shall be of the type specified on the plans and shall meet the requirements of Item 300, "Asphalts, Oils and Emulsions".

(4) **Fertilizer.** Fertilizer, of the type shown on the plans, shall meet the requirements of Item 166, "Fertilizer".

(5) **Water.** Water required for proper compaction, the promotion of plant growth, and/or emulsion dilution shall conform to Item 204, "Sprinkling".

134.3. Construction Methods. Unless otherwise permitted by the Engineer, when backfill material is required to be hauled to or within the project site, the backfill material shall be hauled to the approximate required location prior to placement of the pavement finish surface course. After the pavement finish surface course has been placed, the backfill material shall be spread, compacted, and shaped in accordance with the typical sections.

(1) **Types A and B Backfill.** After the surface course has been placed, the necessary backfill material shall be brought to the approved moisture content, bladed, and compacted as directed by the Engineer. The material shall be shaped to the lines and grades as shown on the plans. After the backfill has been compacted, the roadway sideslopes shall be bladed to a smooth surface conforming to the details indicated on the typical sections or as directed by the Engineer.

(2) **Type C Backfill.** Mulch sodding backfill material shall be placed in a uniform windrow and kept moist as directed by the Engineer.

After the surface course has been placed, the necessary backfill material shall be bladed and compacted in accordance with Subarticle 162.3(8) or as directed by the Engineer. After the backfill has been compacted, the pavement side slopes shall be bladed to a smooth surface conforming to the details indicated on the typical sections or as directed by the Engineer.

(3) **Emulsified Asphalt.** Emulsified asphalt mixture, when shown on the plans, shall be applied following final finishing of the backfill material until the specified amount of mixture has been applied. The rate of application, after dilution, shall be as specified on the plans.

(4) **Fertilizer.** Fertilizer, when shown on the plans, shall be distributed uniformly at the rate specified over the backfilled area following final finishing. After the application of fertilizer, the backfill areas shall be thoroughly moistened to a depth of 100 millimeters or to the maximum depth of the backfill whichever is less.

APPENDIX F GUIDE SCHEDULE OF SAMPLING AND TESTING FOR EMBANKMENT

TABLE F-1.

GUIDE SCHEDULE OF SAMPLING AND TESTING
(Per Contract)
EMBANKMENTS, SUBBASES, AND BASE COURSES

This is a guide for minimum sampling and testing. When necessary for quality control, additional sampling and testing will be required.

English Units

			PROJECT TESTS		INDEPENDENT ASSURANCE TESTS		
MATERIAL OR PRODUCT	TEST FOR	TEST NUMBER	LOCATION OR TIME OF SAMPLING	FREQUENCY OF SAMPLING	LOCATION OR TIME OF SAMPLING	FREQUENCY of SAMPLING	REMARKS
EMBANKMENT	In-place Density (H)	Tex-115-E	As designated by the Engineer	Each 5,000 C.Y. (F)	Same as Project Test	Each 50,000 C.Y. or fraction thereof (B)	Tex-115-E or other approved method
UNTREATED SUBBASE AND BASE COURSES	Gradation (H)	Tex-110-E	During stockpiling oprs, from stockpile, or from windrow (1)	Each 4,000 C.Y. or 6,000 tons	Same as Project Test	One out of 10 Project Tests or fraction thereof (C)	(1) Engineer will select any one of these three locations or any combinations thereof with the provision that at least one of 10 tests will be sampled from the windrow for Gradation, Liquid Limit and Plasticity Index.
	Liquid Limit	Tex-104-E	During stockpiling oprs, from stockpile, or from windrow (1)	Each 4,000 C.Y. or 6,000 tons	Same as Project Test	One out of 20 Project Tests or fraction thereof (C)	
	Plasticity Index	Tex-106-E	During stockpiling oprs, from stockpile, or from windrow (1)	Each 4,000 C.Y. or 6,000 tons	Same as Project Test	One out of 20 Project Tests or fraction thereof (C)	
	Wet Ball Mill	Tex-116-E	During stockpiling oprs. from stockpile, or from windrow	Each 20,000 C.Y. or 25,000 tons			When a stockpile is to be sampled that has not been built in horizontal layers, sampling will be one test for each 4,500 C.Y. or 6,000 tons.
	Triaxial	Tex-117-E	During stockpiling oprs. from stockpile, or from windrow	Each 20,000 C.Y. or 25,000 tons (D)			Triaxial tests are not a field laboratory function. When a stockpile is to be sampled that was not built in horizontal layers, sampling will be one test for each 12,000 C.Y. or 16,000 tons.
	Compaction (H)	Tex-115-E	As designated by the Engineer	Each 3,000 lin. ft. per course per travel-way (A)	Same as Project Test	One out of 10 Project Tests or fraction thereof (C)	Tex-115-E or other approved method
	Thickness (H)		As designated by the Engineer	One depth per 3,000 lin. ft. per travel-way (A) (E)	Same as Project Test	One total depth per travel-way per two miles or fraction thereof (A)(C)	If payment is by the S.Y. frequency shall be as called for in the governing specification.

(continued…)

GUIDE SCHEDULE OF SAMPLING AND TESTING
(Per Contract)
EMBANKMENTS, SUBBASES, AND BASE COURSES (Cont.)

This is a guide for minimum sampling and testing. When necessary for quality control, additional sampling and testing will be required.

English Units

MATERIAL OR PRODUCT		TEST FOR	TEST NUMBER	PROJECT TESTS: LOCATION OR TIME OF SAMPLING	PROJECT TESTS: FREQUENCY OF SAMPLING	INDEPENDENT ASSURANCE TESTS: LOCATION OR TIME OF SAMPLING	INDEPENDENT ASSURANCE TESTS: FREQUENCY OF SAMPLING	REMARKS
TREATED SUBBASE AND BASE COURSES	Base Material	As shown above for untreated base (H)		As designated by the Engineer prior to the addition of a stabilizer	As shown above for untreated base	Same as Project Test	As shown above for untreated base	When central mix site or plant is used, windrow sampling may be waived.
	Lime	Compliance with Item 264 (I)	Tex-600-J	During delivery to project	TY A; 1 Per Project (I) TY B ea., 200 tons or fraction thereof TY C; 1 Per Project (I)			On projects requiring less than 50 tons, material from CSTM approved sources may be accepted on the basis of Producer's Certification without sampling.
	Cement	Compliance with the Std. Specifications & Spl. Provisions	AASHTO M 85	Railroad car, truck or cement bins	Each 2,000 bbls. for each type and brand			Each brand and each type to be sampled and tested separately. Sampling will be waived when source is certified by CSTM.
	Asphalt	Compliance with Item 300	Tex-500-C etc.	Sampled, tested and approved by CSTM				
	Fly Ash	Compliance with Dept. Matl. Spec. D9-8900	Tex-733-I	Sampled, tested and approved by CSTM				
	Complete Mixture	Pulverization	Tex-101-E Part III	Roadway; after pulverization	As necessary for control (G)			Where required to control degree of pulverization
		In-place Density (H)	Tex-115-E	As designated by the Engineer	Each 3,000 lin. ft. per course per travel-way (A)	Same as Project Test	One out of 10 Project Tests or fraction thereof (C)	Tex-115-E or other approved method
		Thickness (H)		As designated by the Engineer	Each 3,000 lin. ft. per course per travel-way (A) (E)	Same as Project Test	One total depth per travel-way per two miles or fraction thereof (A)(C)	When base is measured by the square yard the frequency will be as called for in the governing specification.

(A) Travel-way is defined, for sampling & testing only, as total width of a travel facility that is not separated from other parallel travel facilities by a median, ditch, etc.

(B) Independent Assurance Tests are not required for a contract quantity of less than 25,000 C.Y.

(C) Independent Assurance Tests are not required for a contract quantity resulting in less than 6 acceptance tests.

(D) When base material is from a source where the District has a record of satisfactory triaxial results, the frequency of testing may be reduced to one per 30,000 C.Y. or 40,000 tons. If any one test falls below the minimum value required, the frequency of testing will return to that required by this guide.

(E) Not required where survey grade control documents compliance.

(F) Or approximately one foot compacted depth per lift as approved and directed by the Engineer.

(G) At the beginning of the project, one test will be made for each 4,000 C.Y. or 6,000 tons until such time as the Engineer is satisfied that acceptable pulverization results are being obtained.

(H) When a non-exempt federal-aid project test fails but the product is accepted, the reasons for acceptance should be documented on the Letter of Certification of Materials Used.

(I) For Types A and C lime, sources not on the TxDOT Quality Monitoring Program will be sampled each 200 and 150 tons respectively.

APPENDIX G NUMERICAL SIMULATION RESULTS

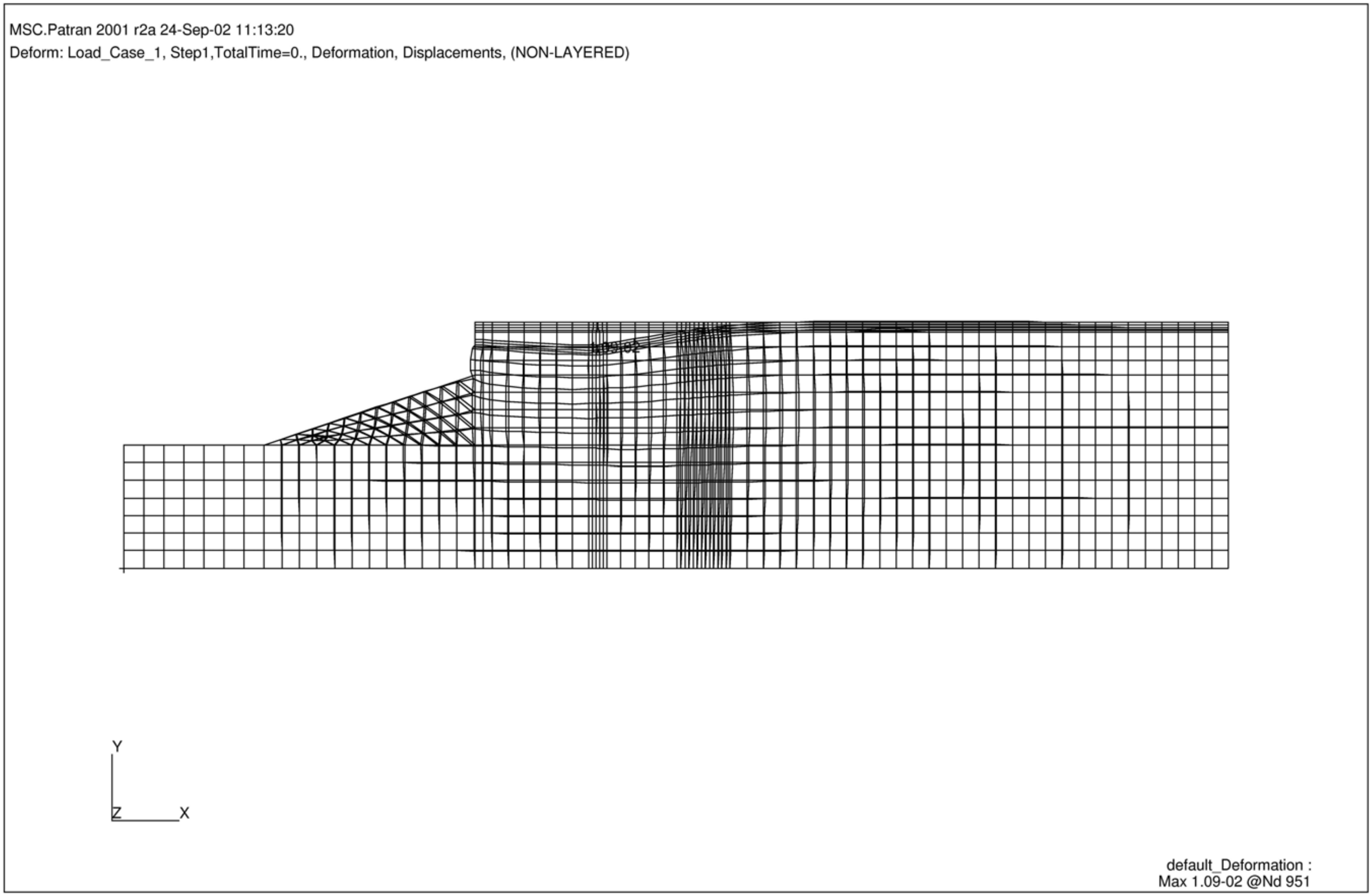


Figure G-1. Load Case 1, No Wall, E=2,500 kPa in Zone 3.

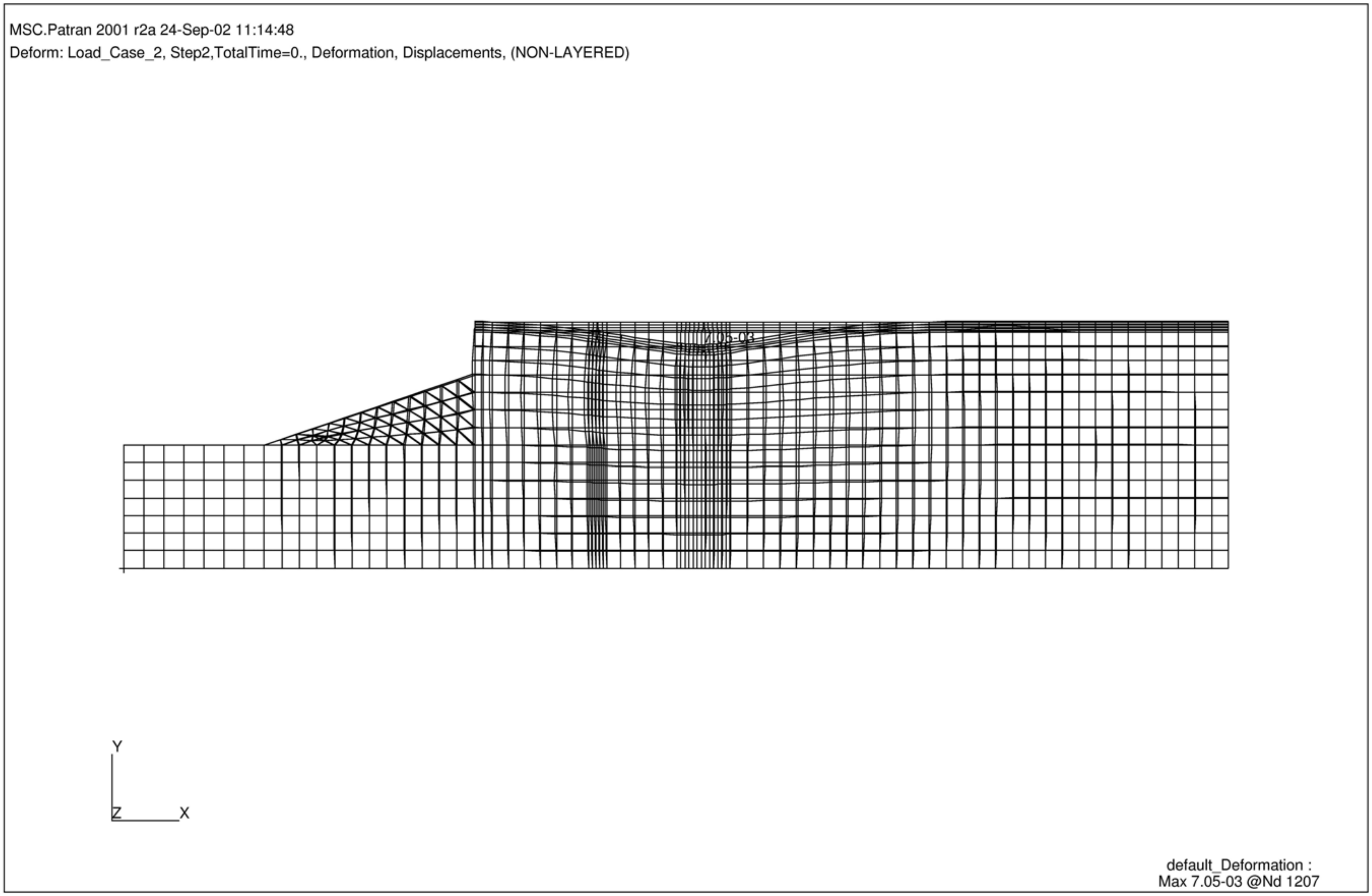


Figure G-2. Load Case 2, No Wall, E=2,500 kPa in Zone 3.

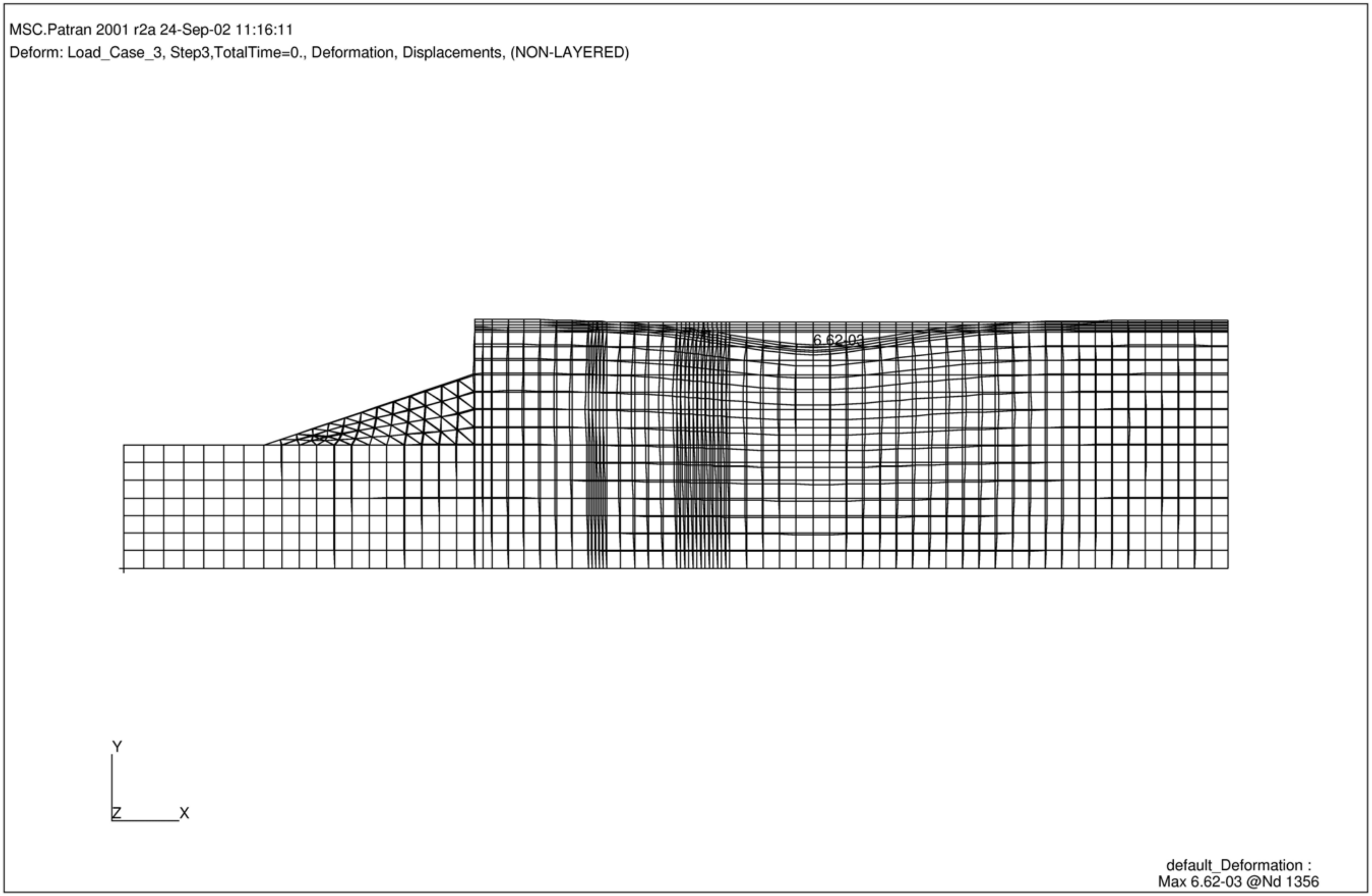


Figure G-3. Load Case 3, No Wall, E=2,500 kPa in Zone 3.

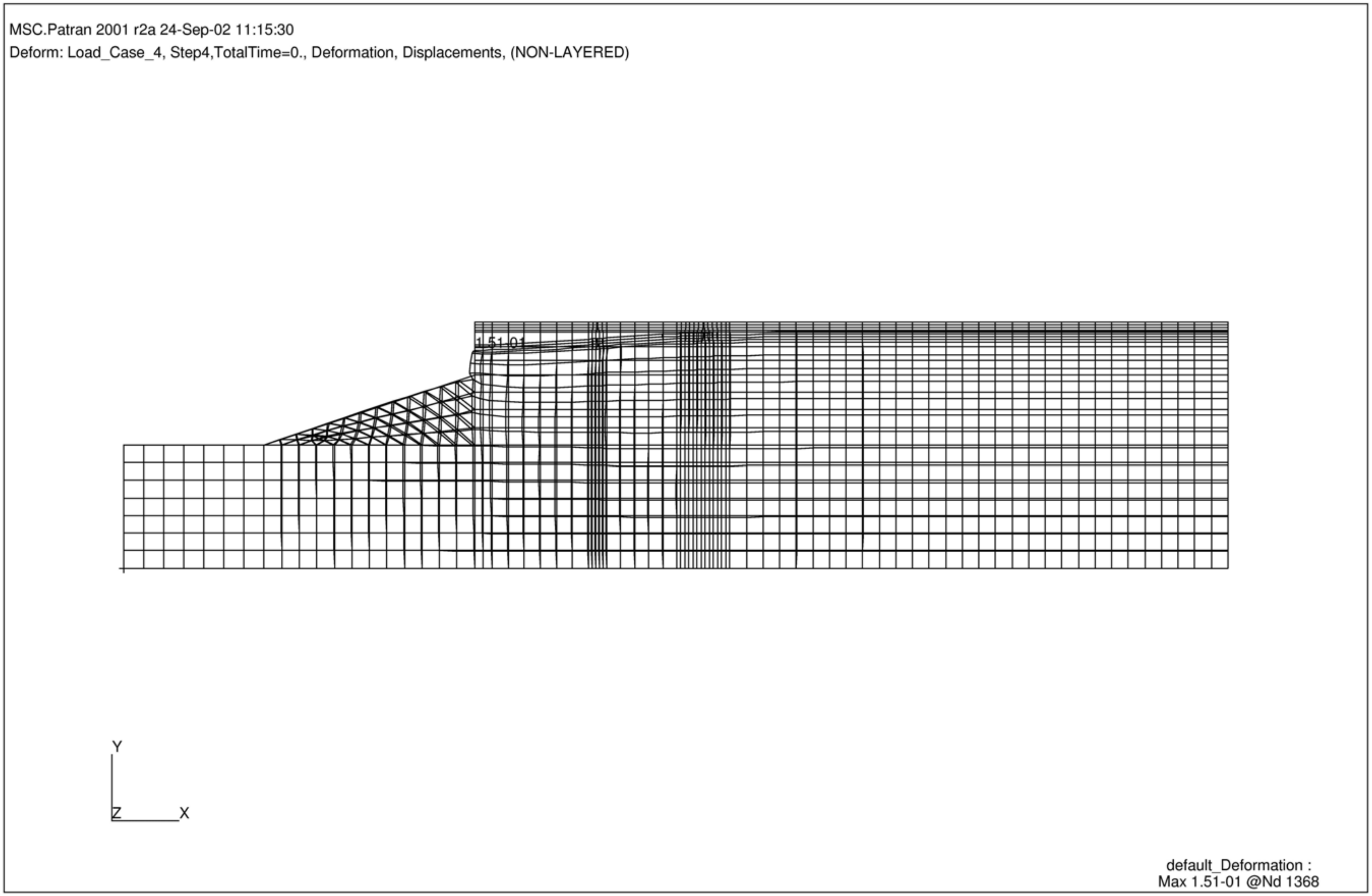


Figure G-4. Load Case 4, No Wall, E=2,500 kPa in Zone 3.

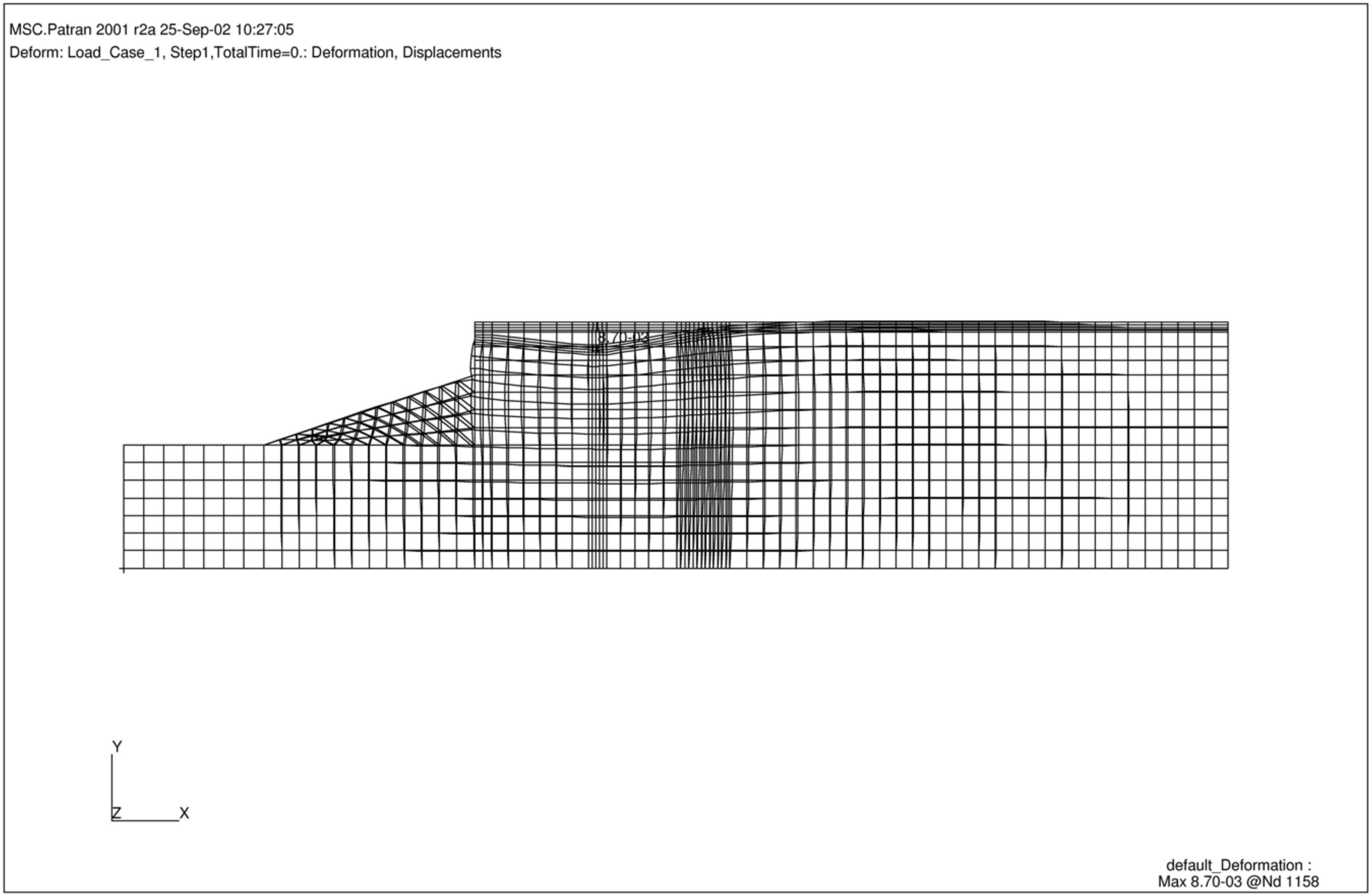


Figure G-5. Load Case 1, No Wall, E=5,000 kPa in Zone 3.

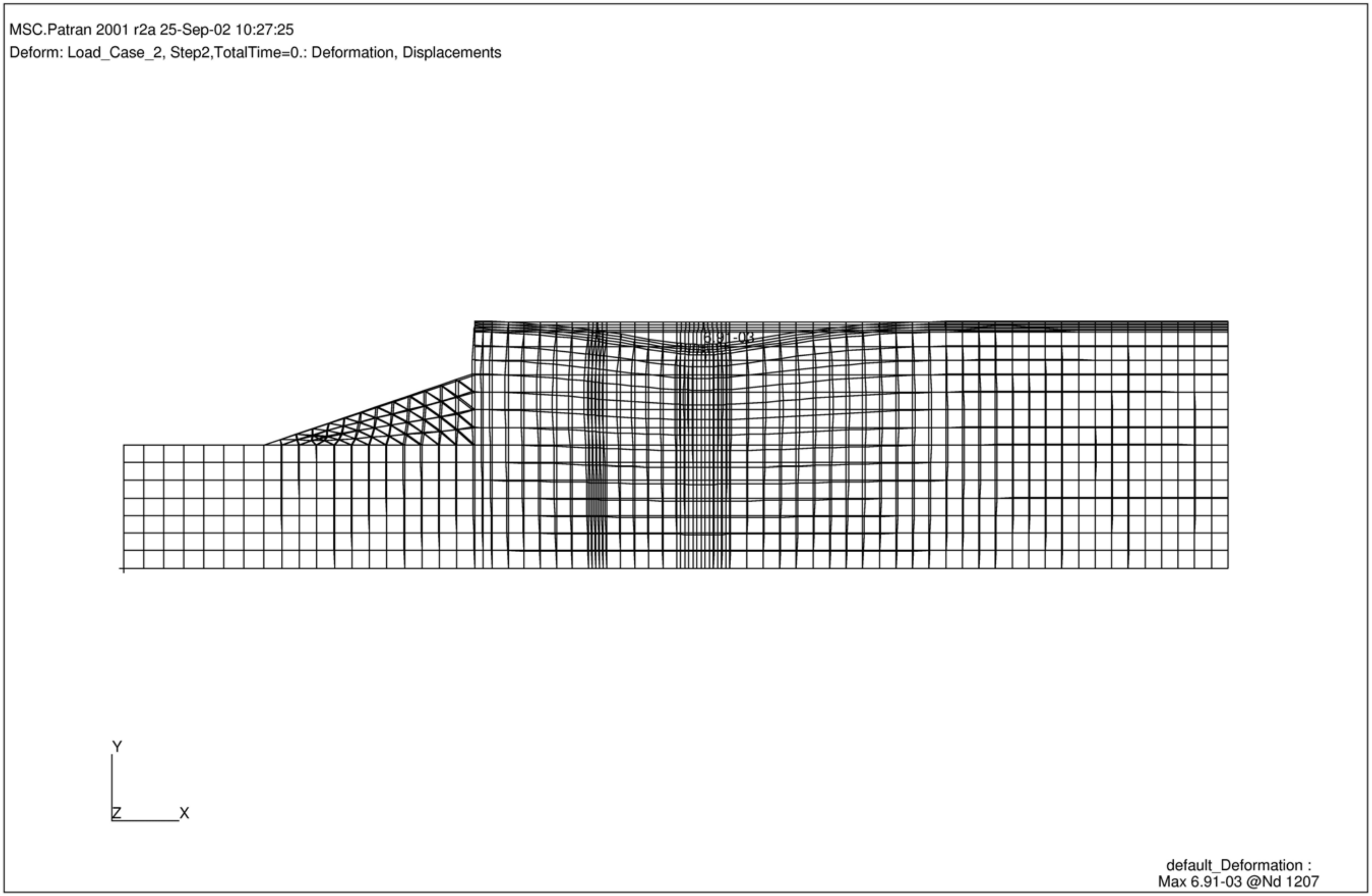


Figure G-6. Load Case 2, No Wall, E=5,000 kPa in Zone 3.

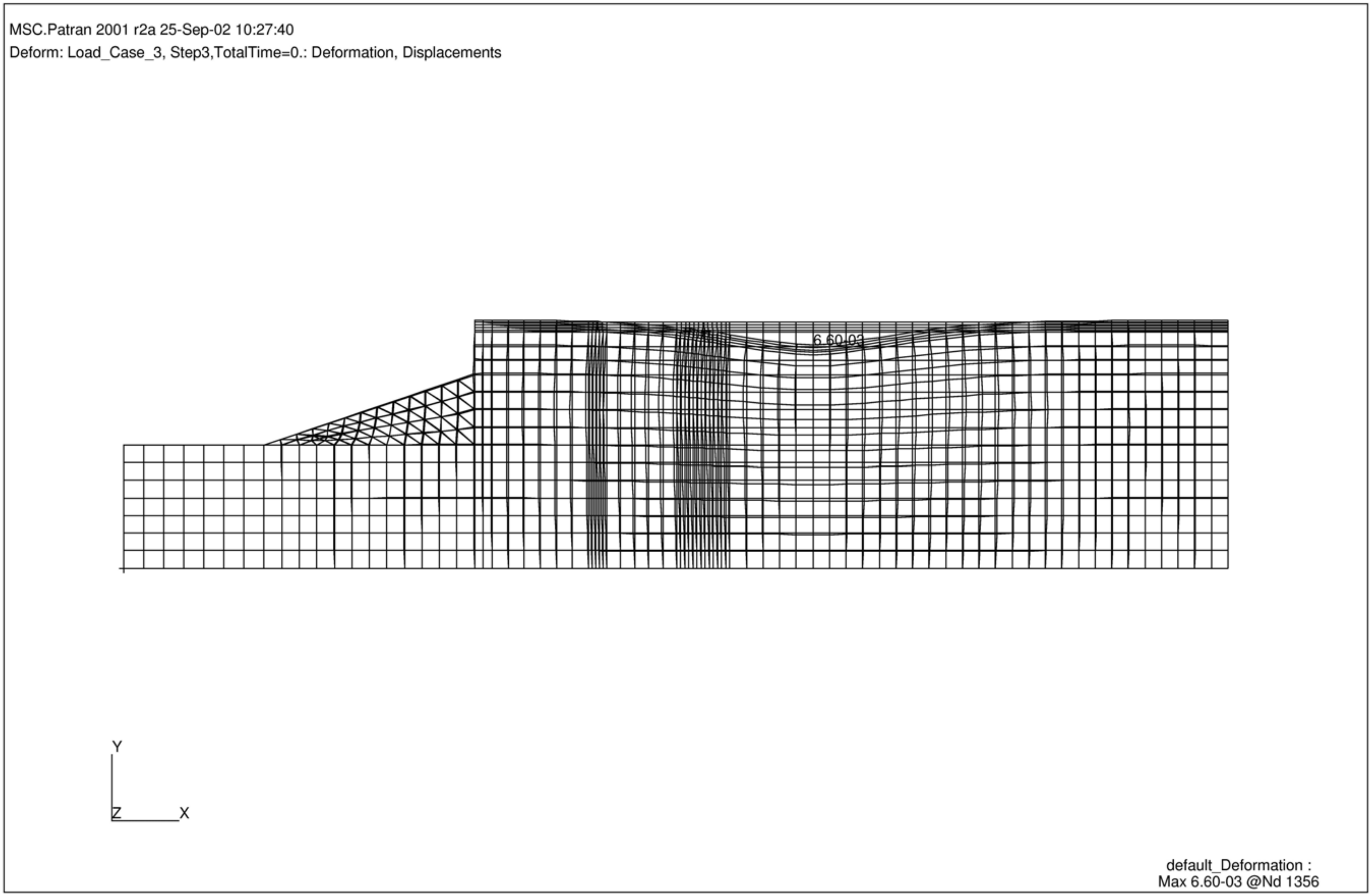


Figure G-7. Load Case 3, No Wall, E=5,000 kPa in Zone 3.

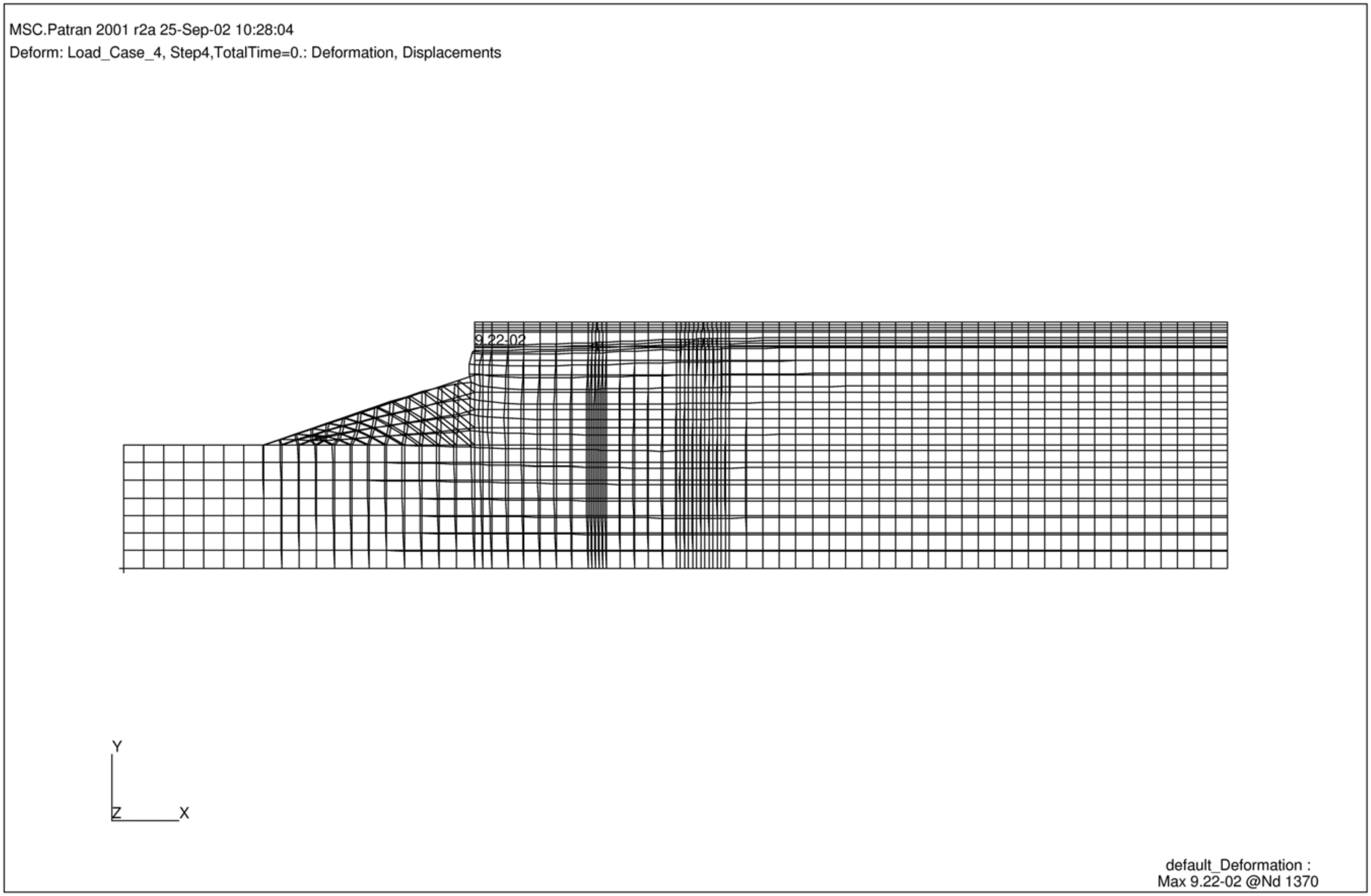


Figure G-8. Load Case 4, No Wall, E=5,000 kPa in Zone 3.

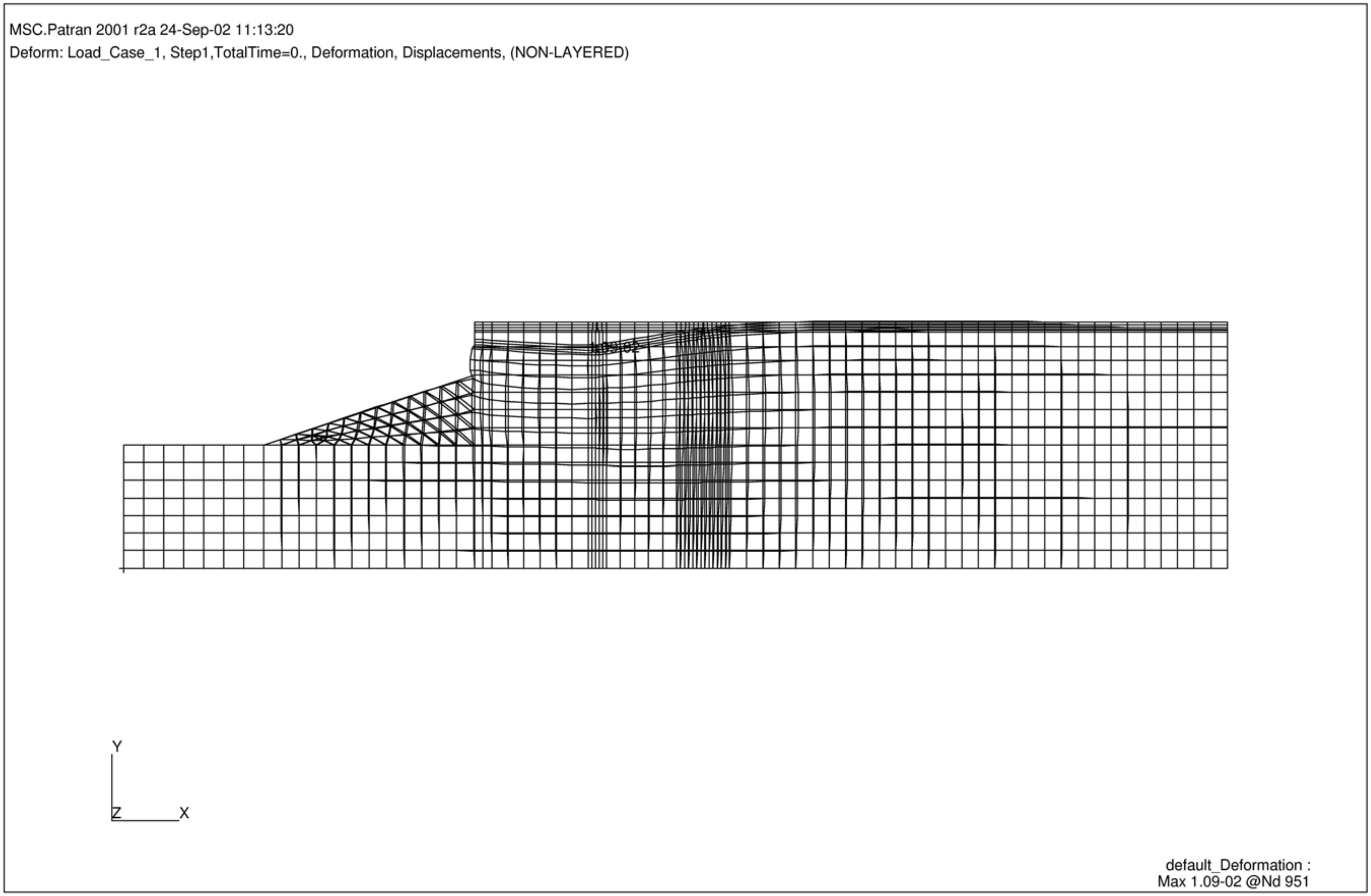


Figure G-9. Load Case 1, No Wall, E=10,000 kPa in Zone 3.

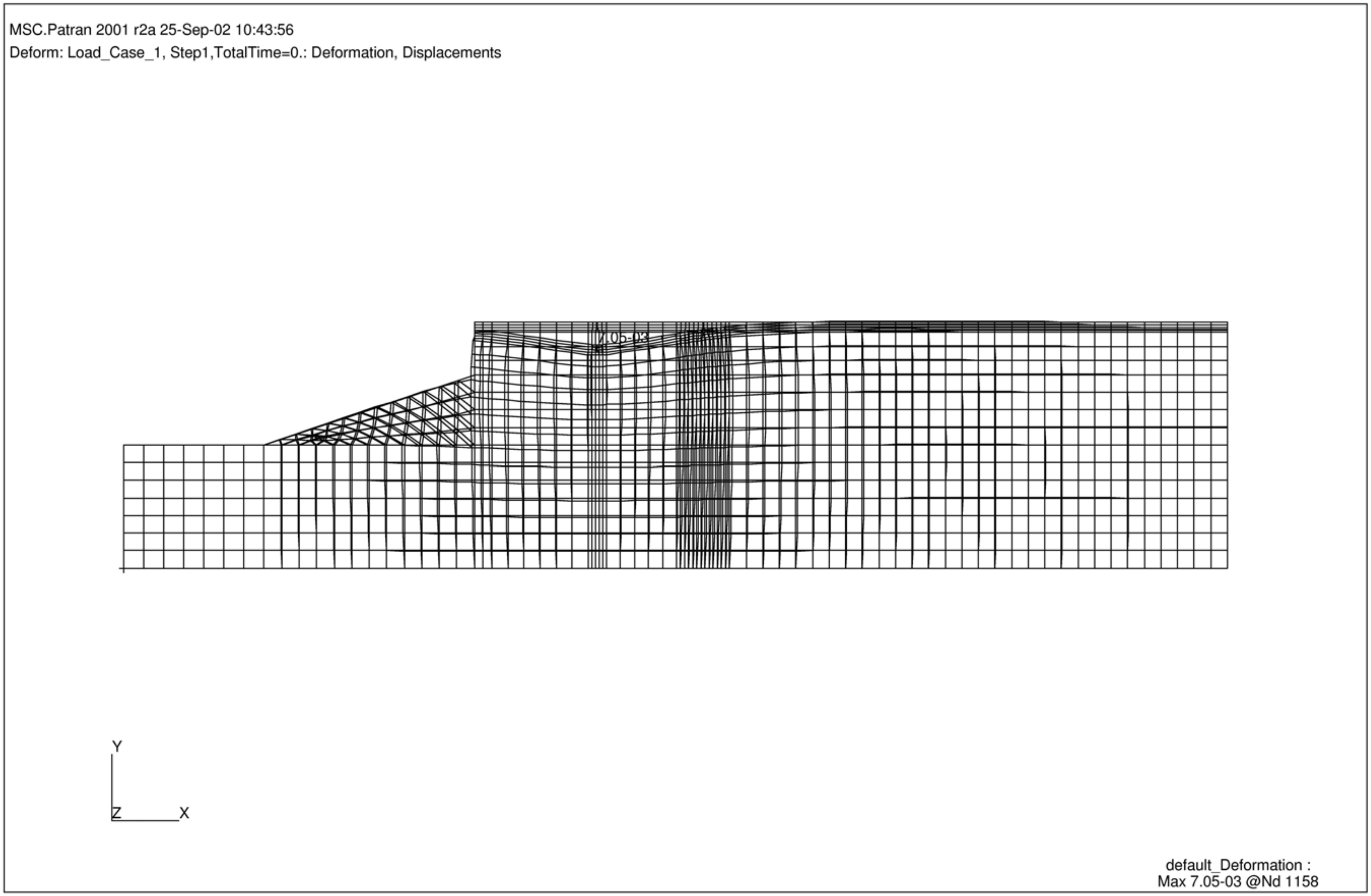


Figure G-10. Load Case 2, No Wall, E=10,000 kPa in Zone 3.

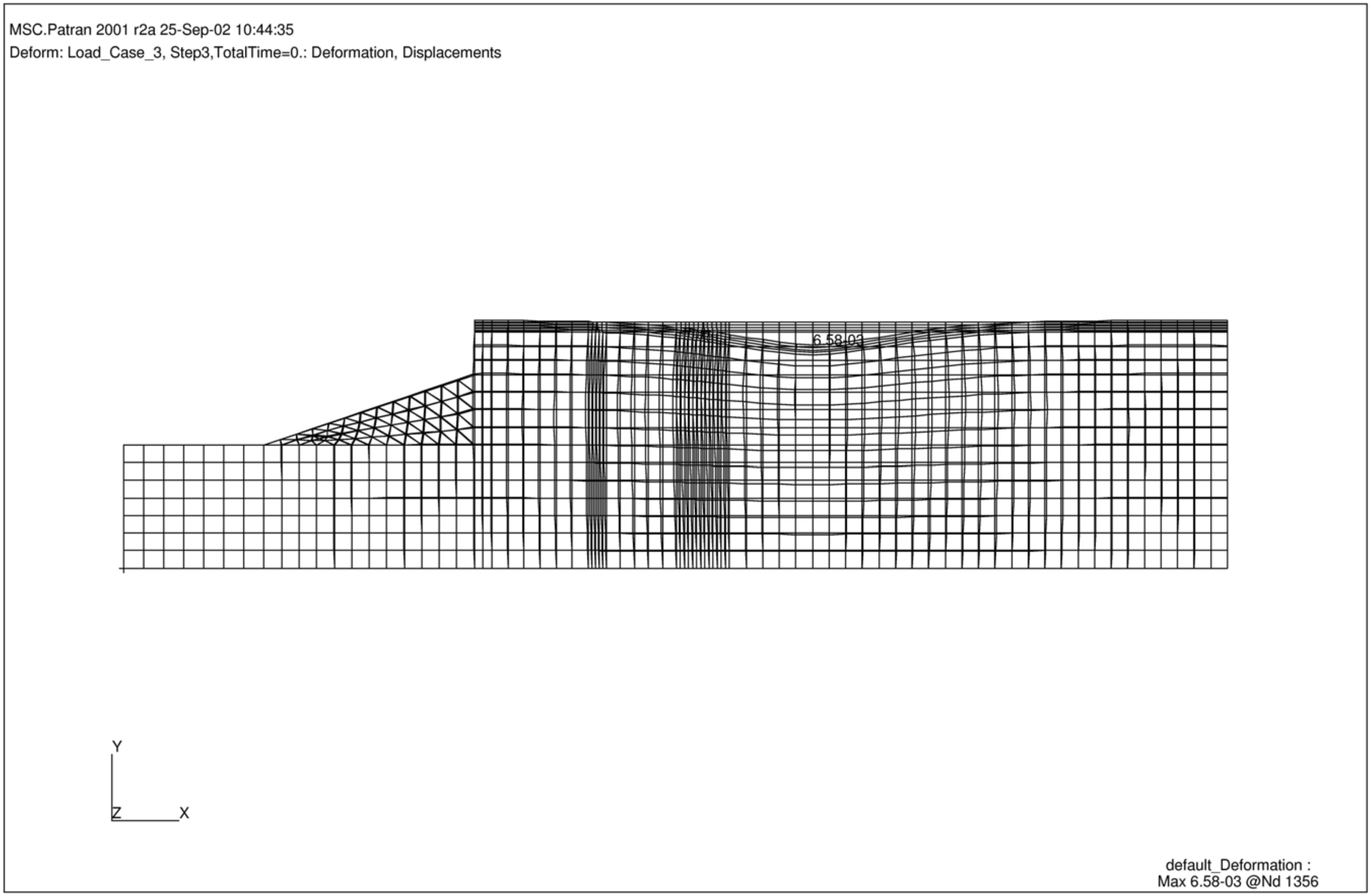


Figure G-11. Load Case 3, No Wall, E=10,000 kPa in Zone 3.

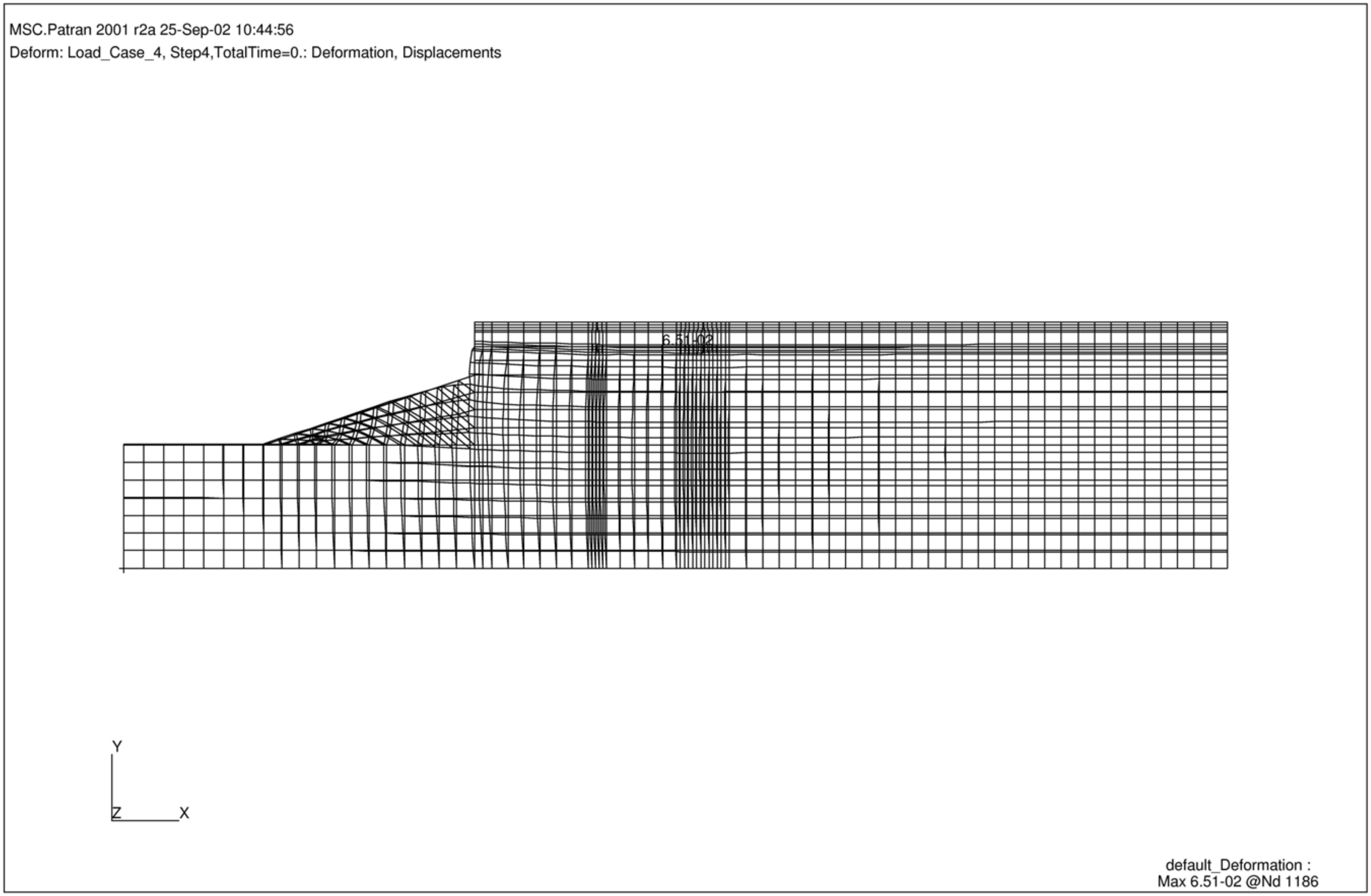


Figure G-12. Load Case 4, No Wall, E=10,000 kPa in Zone 3.

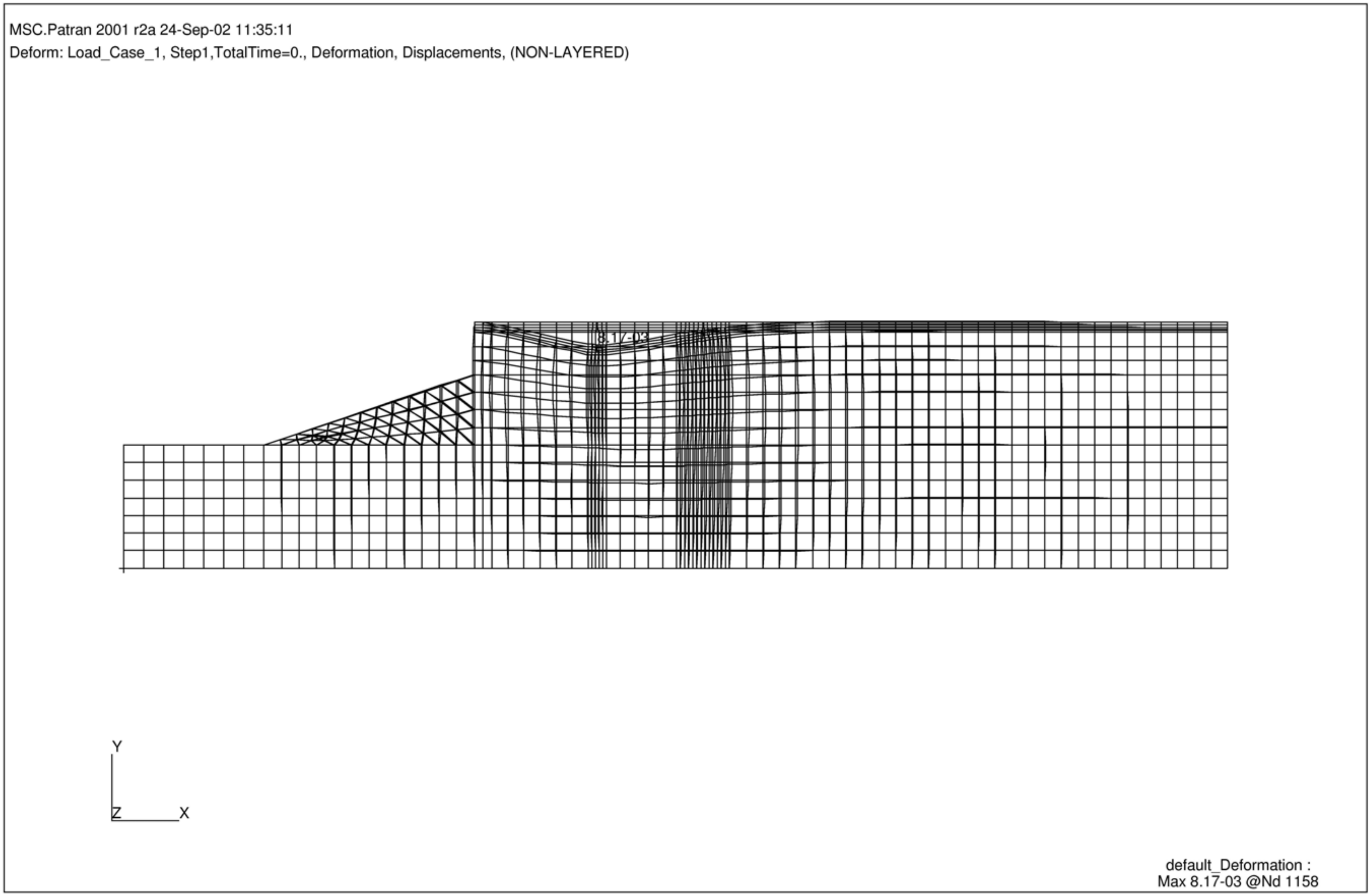


Figure G-13. Load Case 1, 0.5 m Wall, E=2,500 kPa in Zone 3.

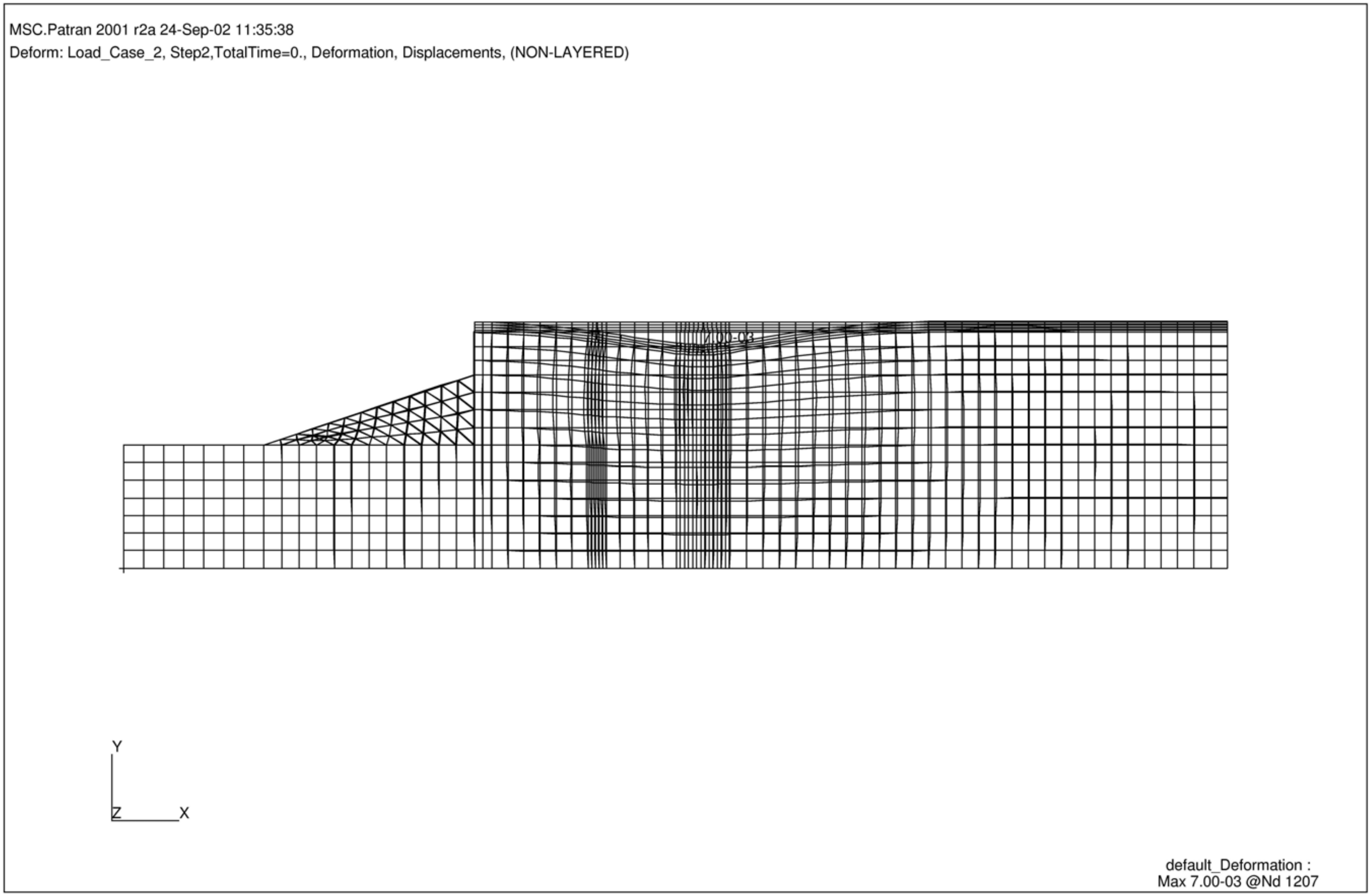


Figure G-14. Load Case 2, 0.5 m Wall, E=2,500 kPa in Zone 3.

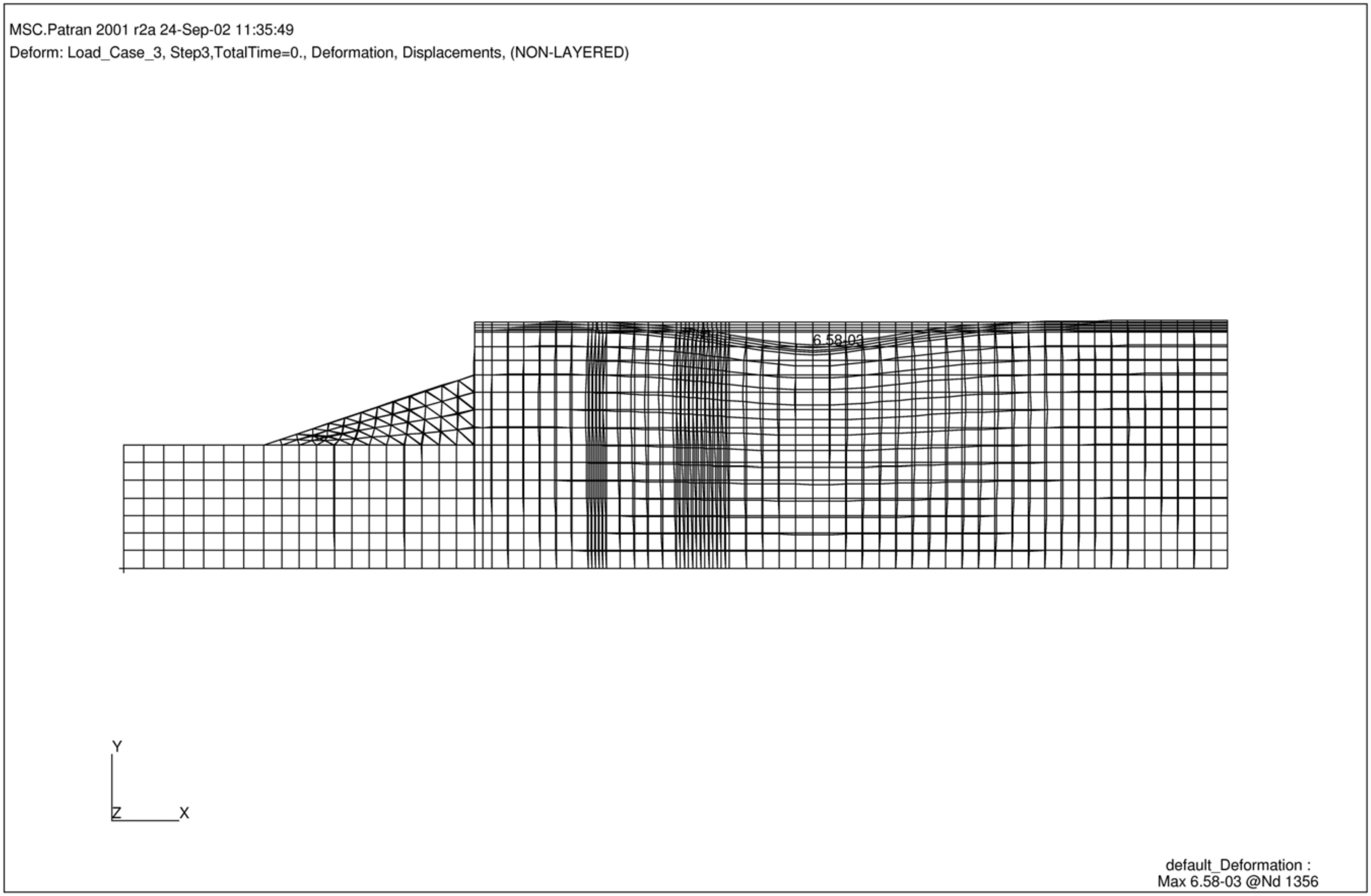


Figure G-15. Load Case 3, 0.5 m Wall, E=2,500 kPa in Zone 3.

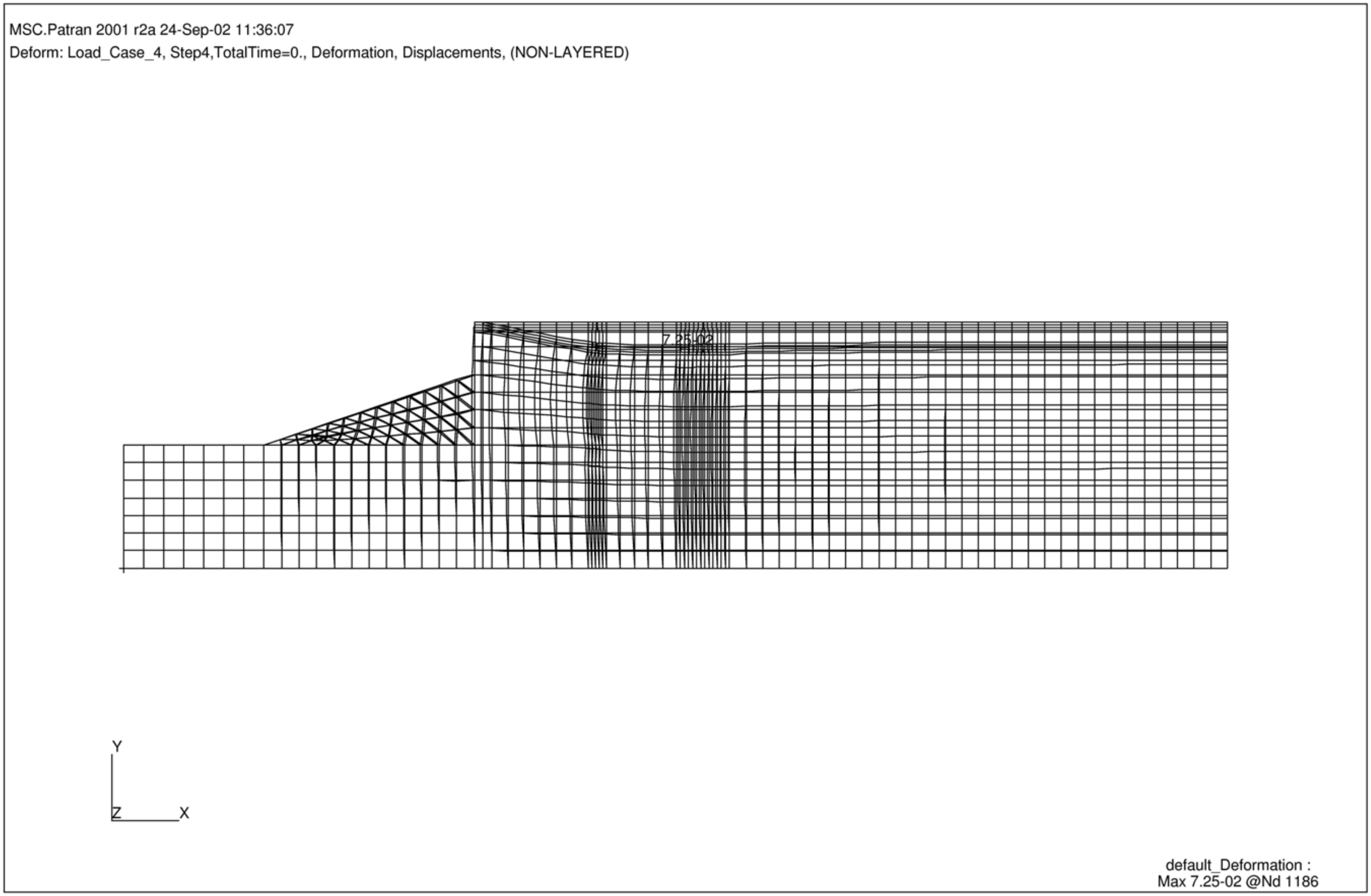


Figure G-16. Load Case 4, 0.5 m Wall, E=2,500 kPa in Zone 3.

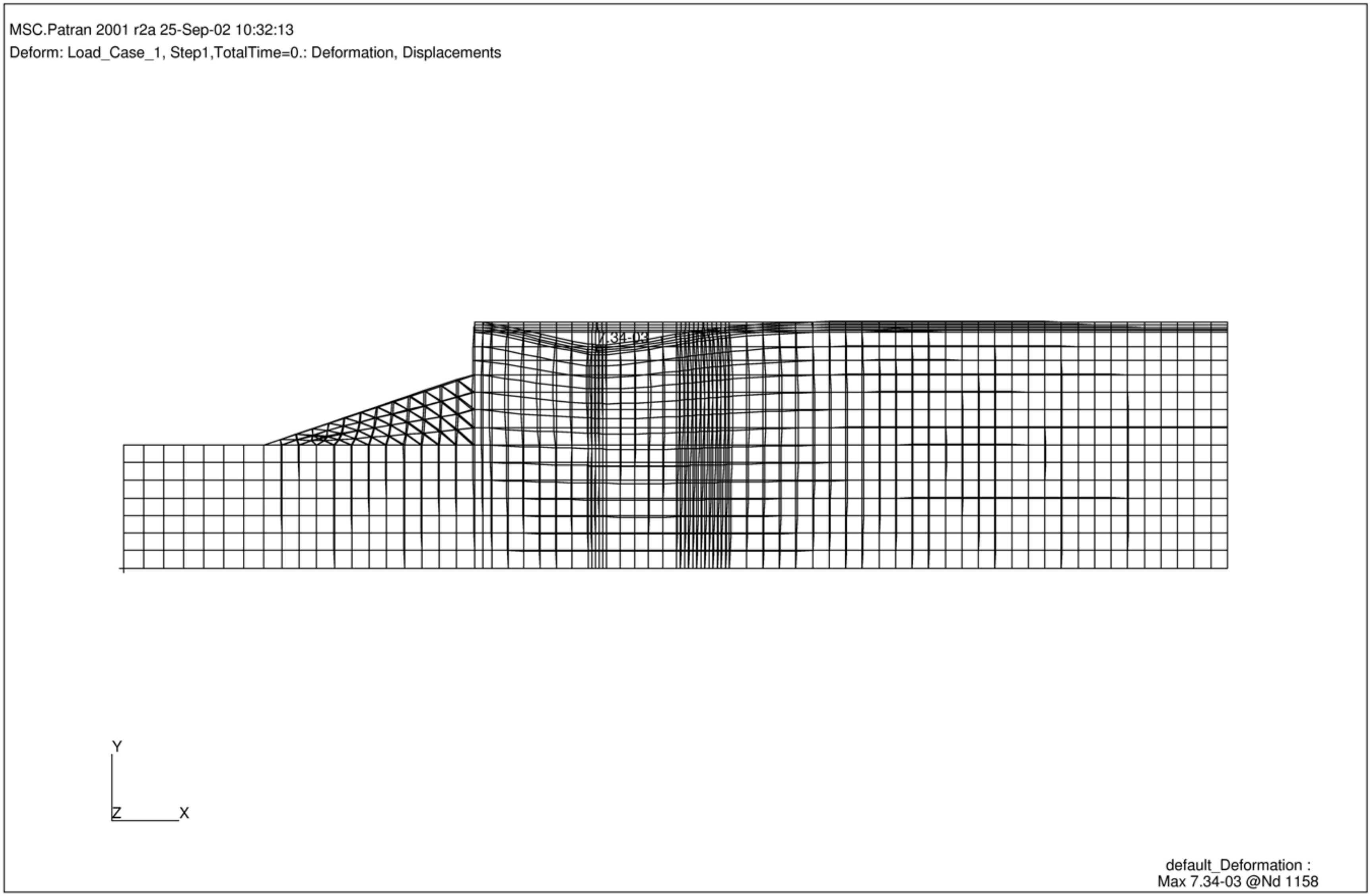


Figure G-17. Load Case 1, 0.5 m Wall, E=5,000 kPa in Zone 3.

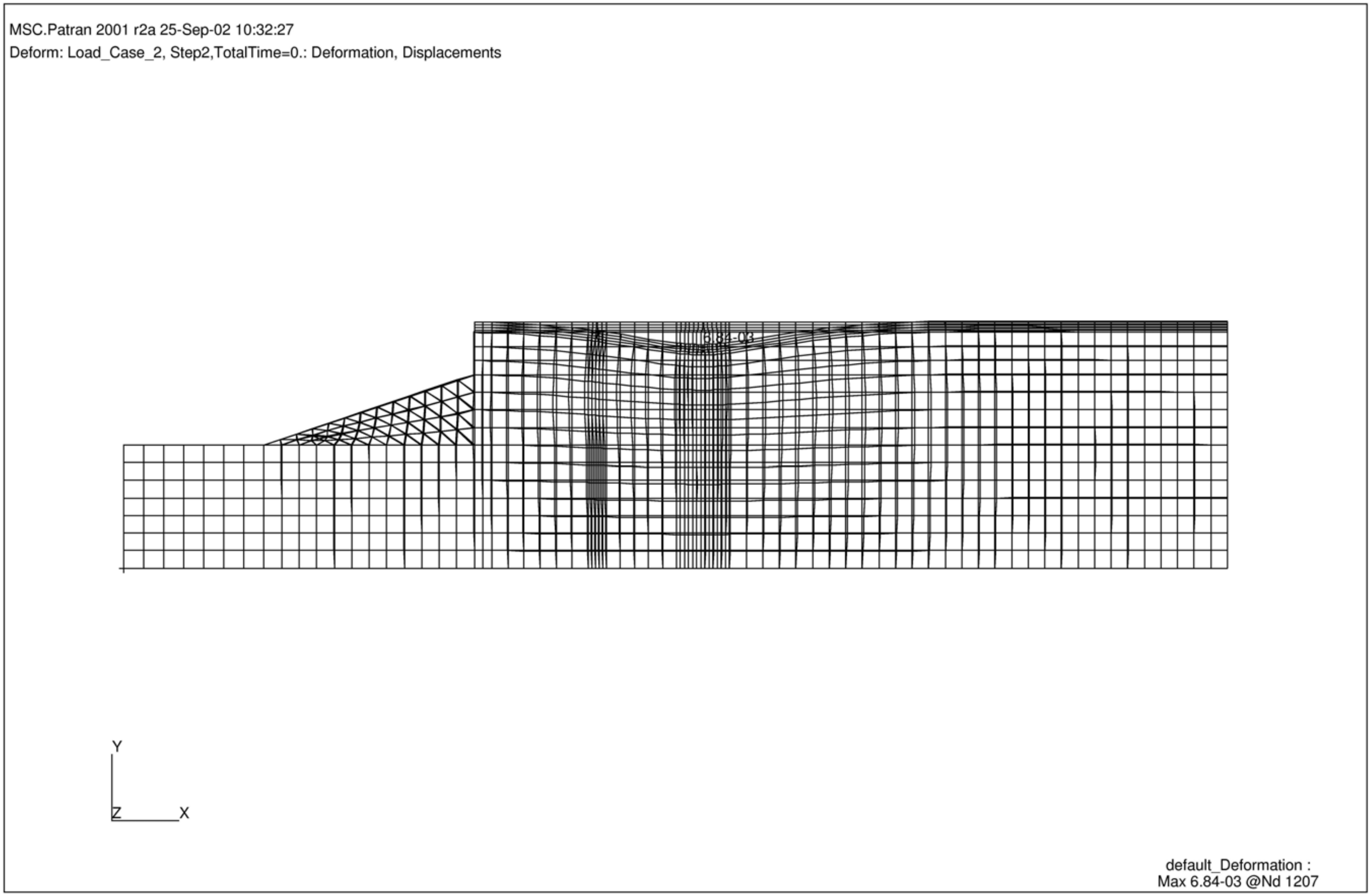


Figure G-18. Load Case 2, 0.5 m Wall, E=5,000 kPa in Zone 3.

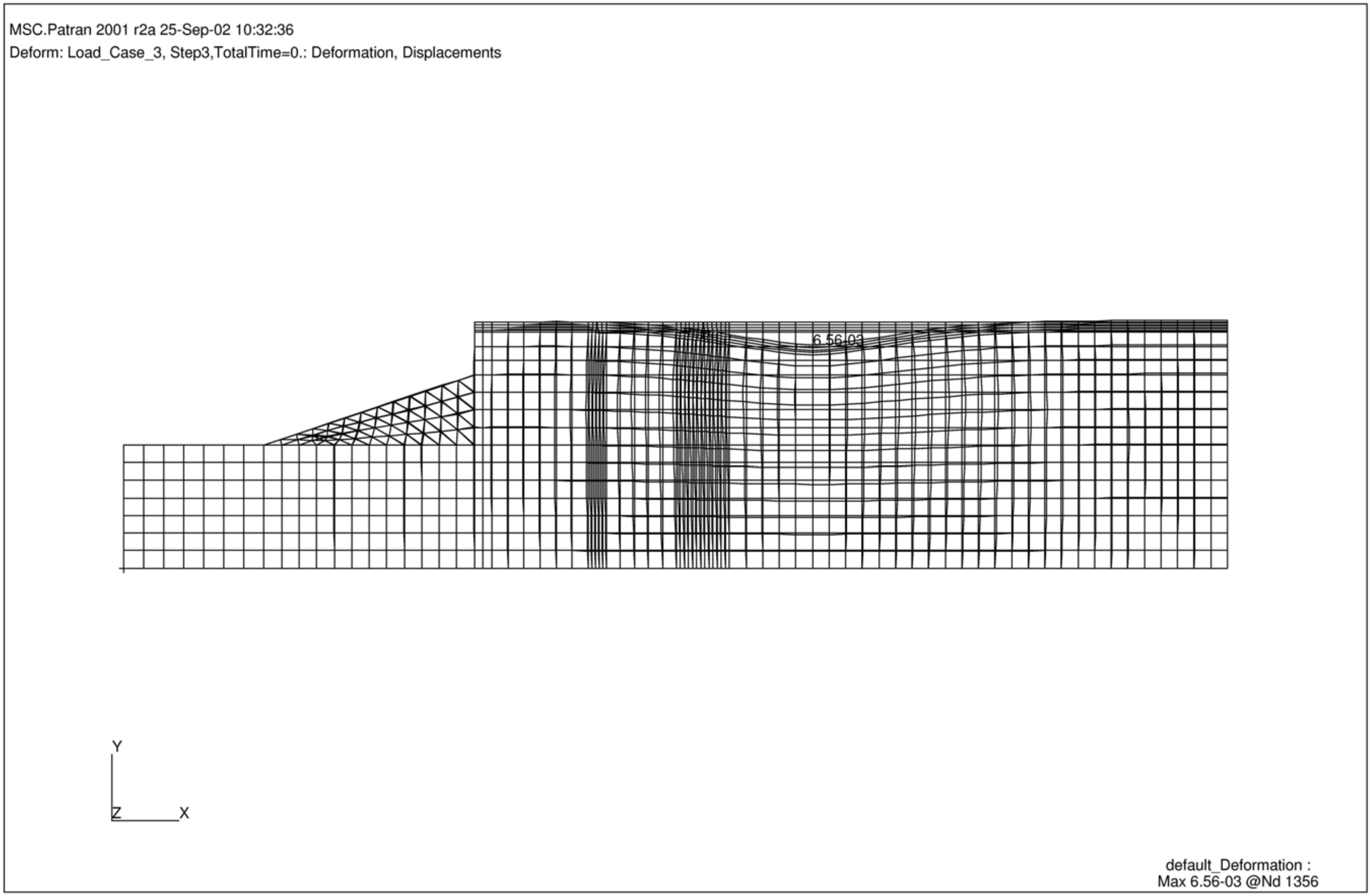


Figure G-19. Load Case 3, 0.5 m Wall, E=5,000 kPa in Zone 3.

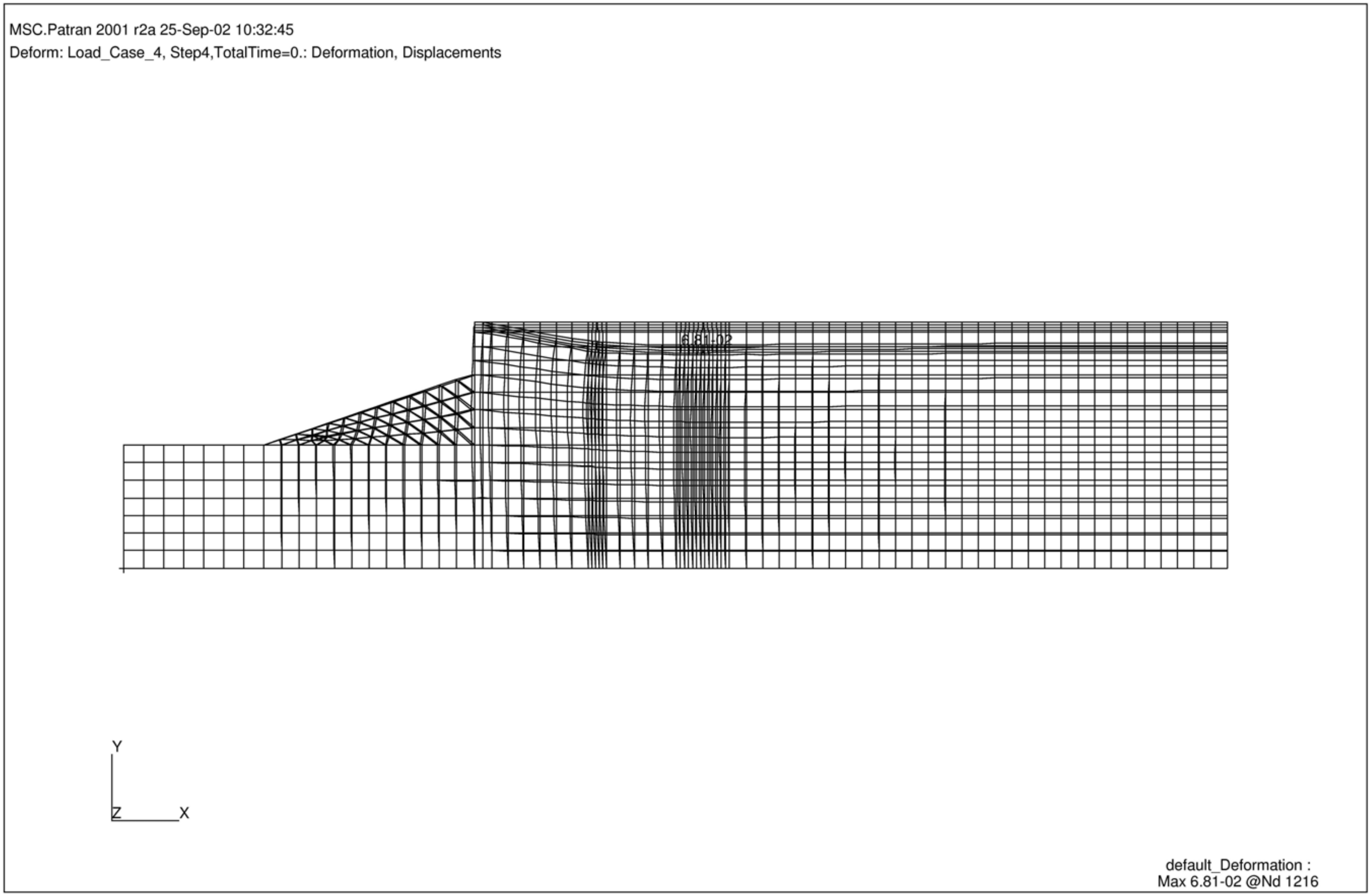


Figure G-20. Load Case 4, 0.5 m Wall, E=5,000 kPa in Zone 3.

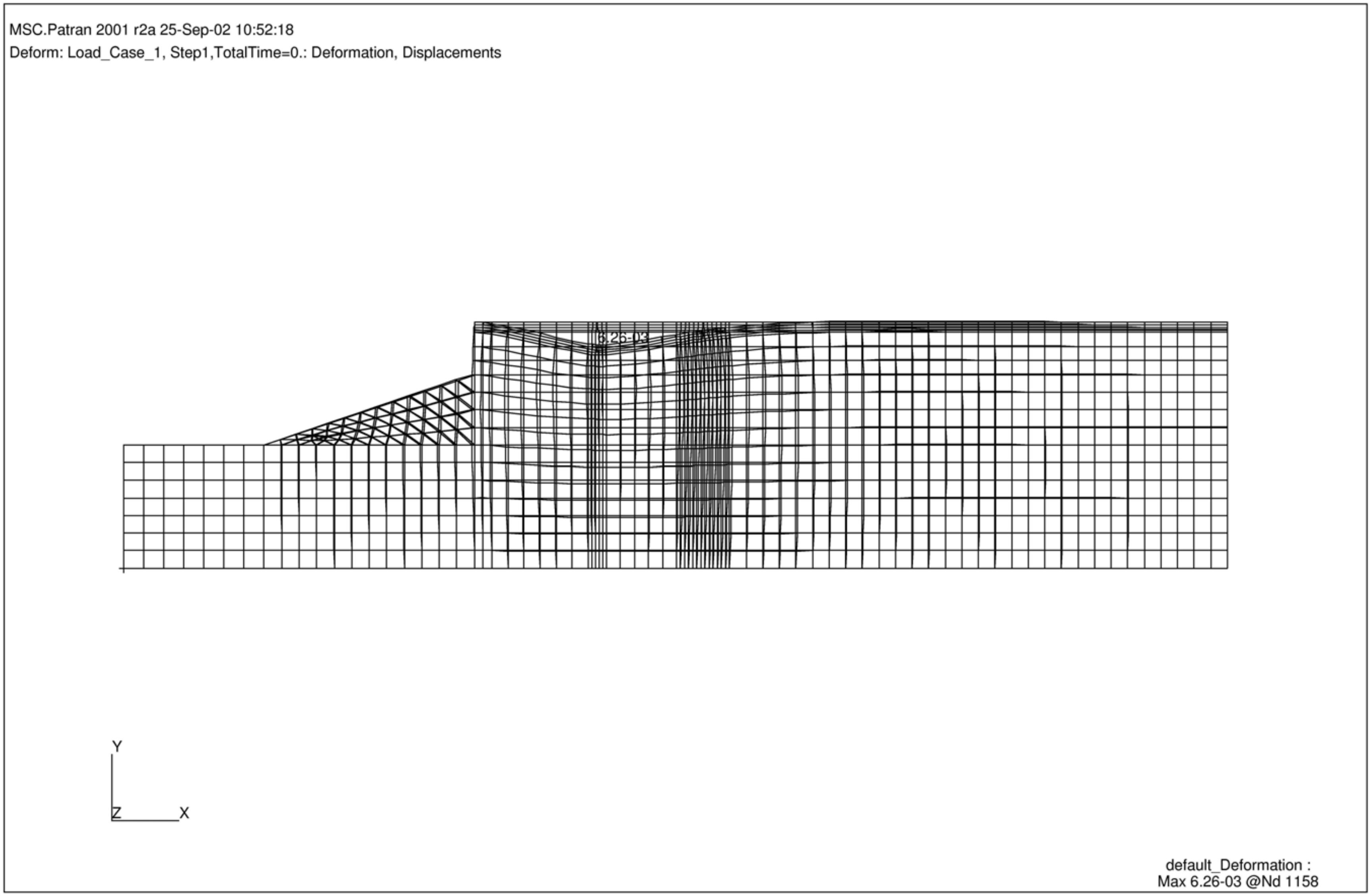


Figure G-21. Load Case 1, 0.5 m Wall, E=10,000 kPa in Zone 3.

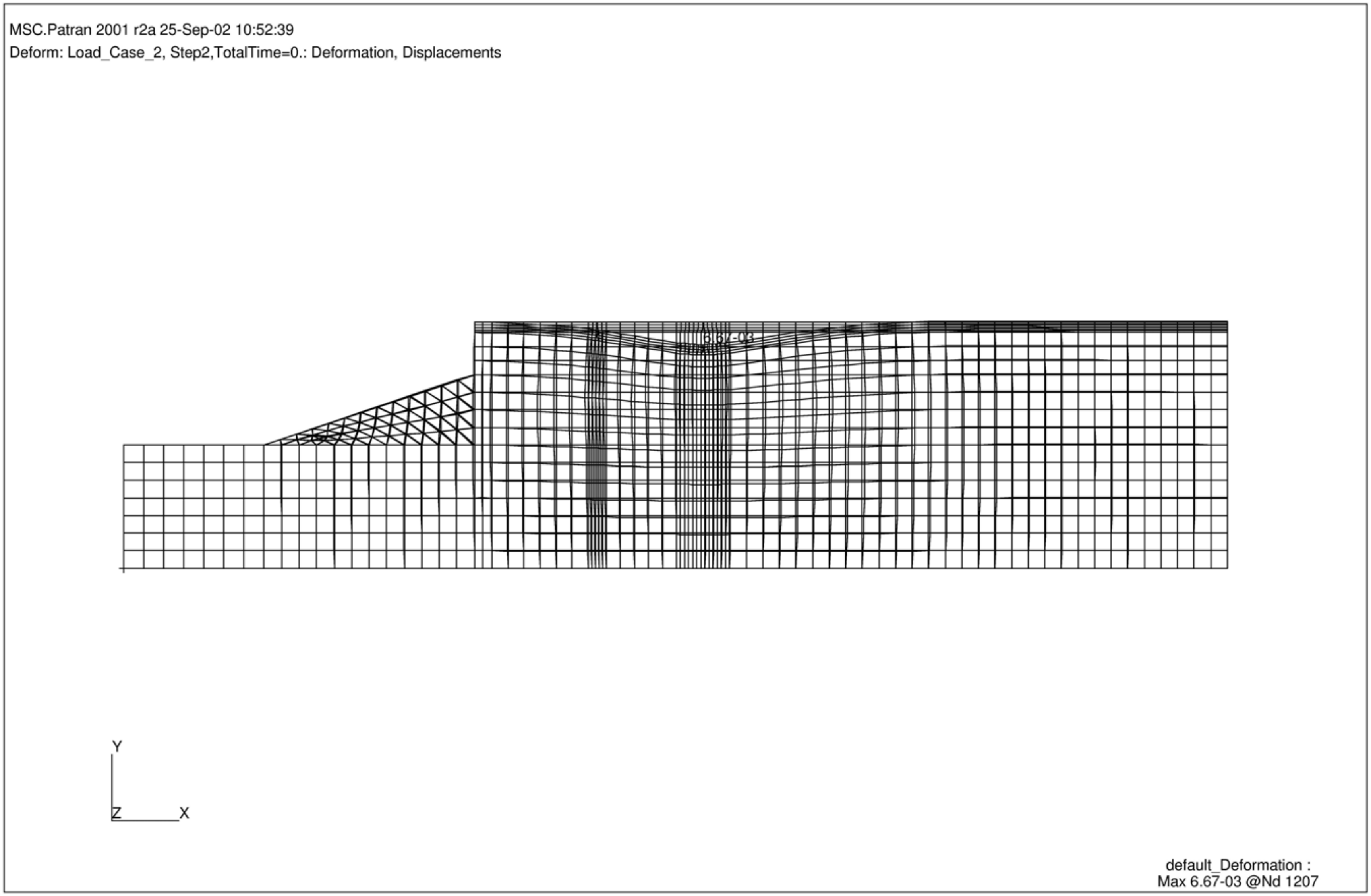


Figure G-22. Load Case 2, 0.5 m Wall, E=10,000 kPa in Zone 3.

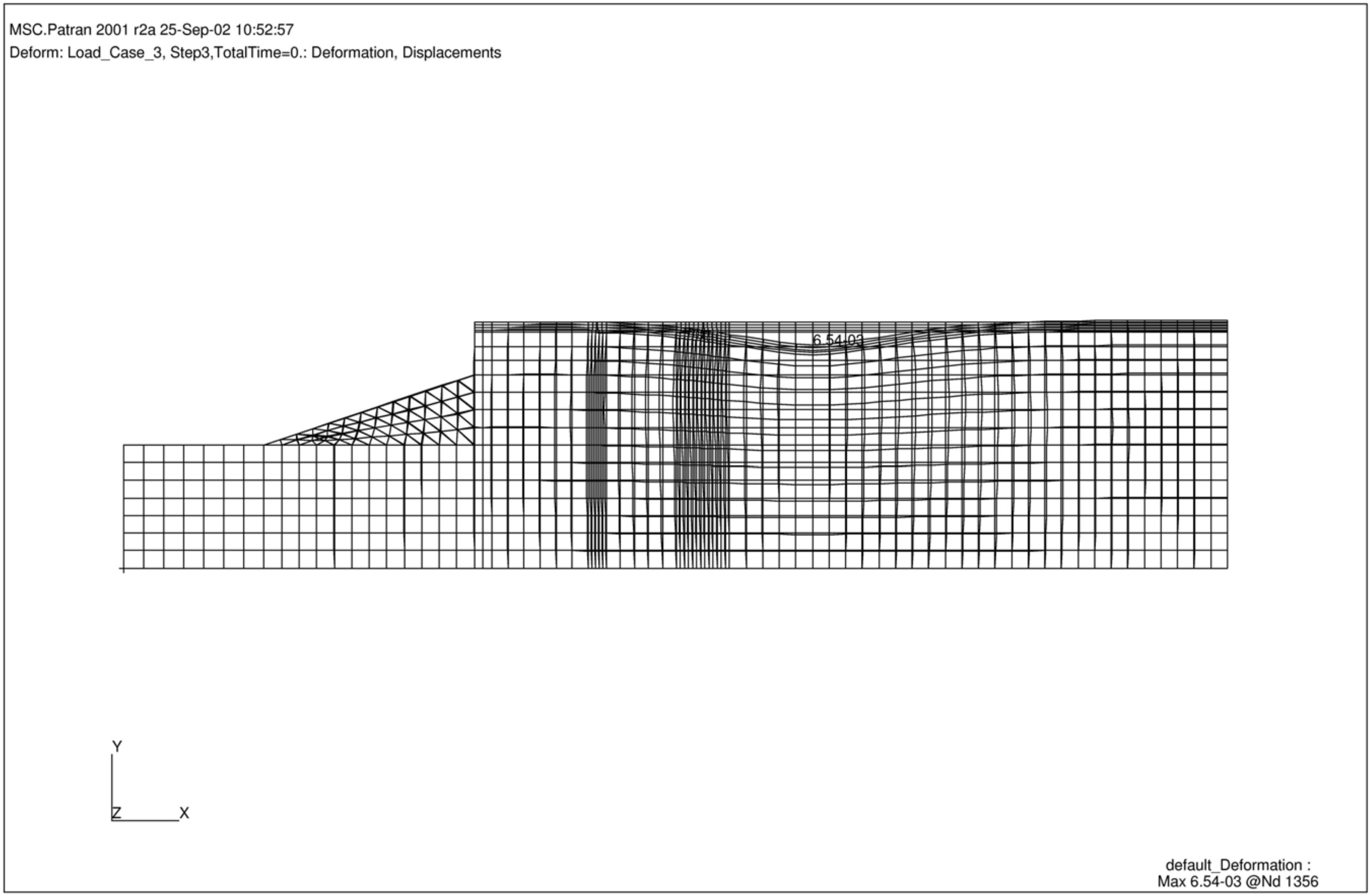


Figure G-23. Load Case 3, 0.5 m Wall, E=10,000 kPa in Zone 3.

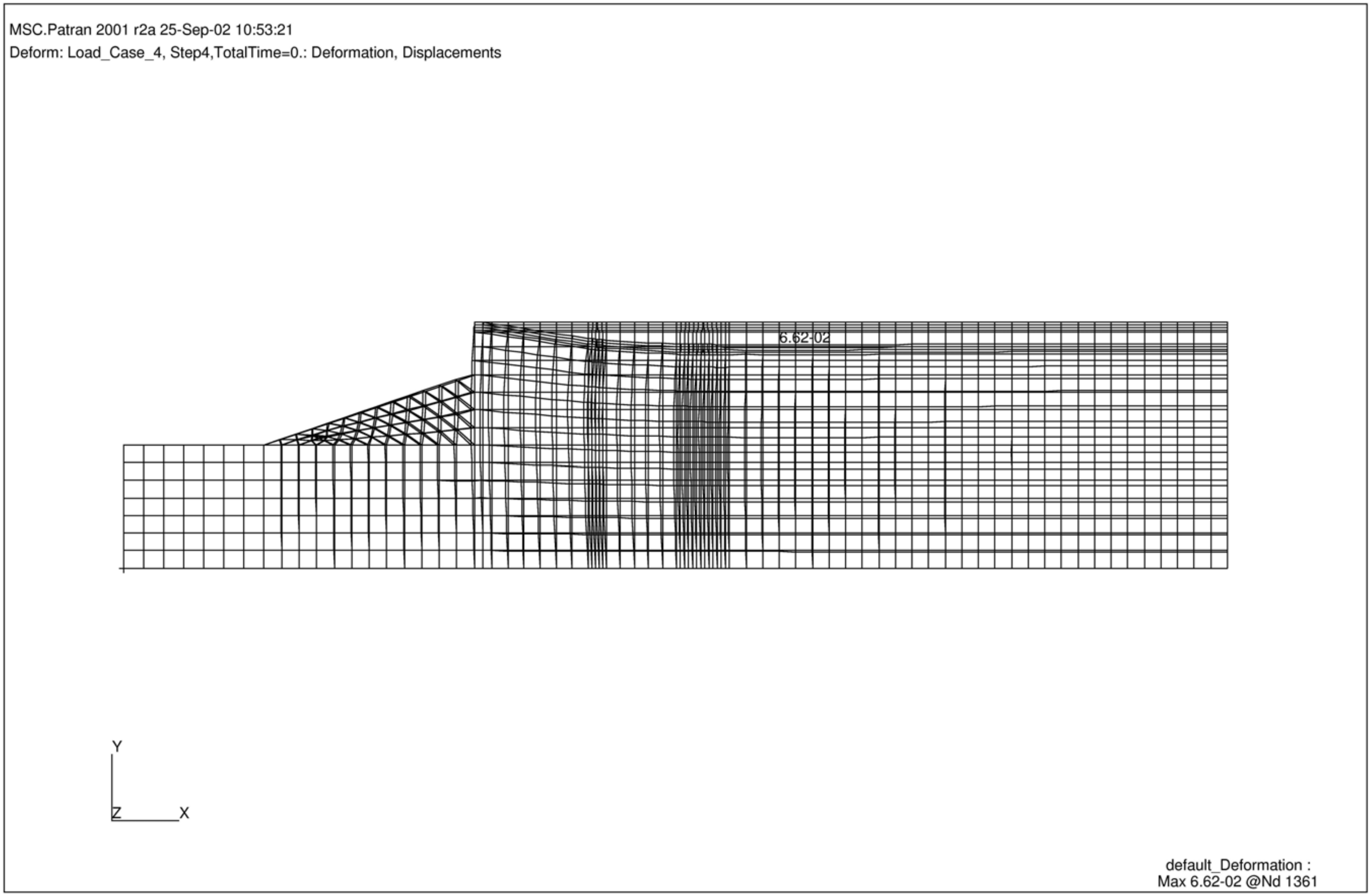


Figure G-24. Load Case 4, 0.5 m Wall, E=10,000 kPa in Zone 3.

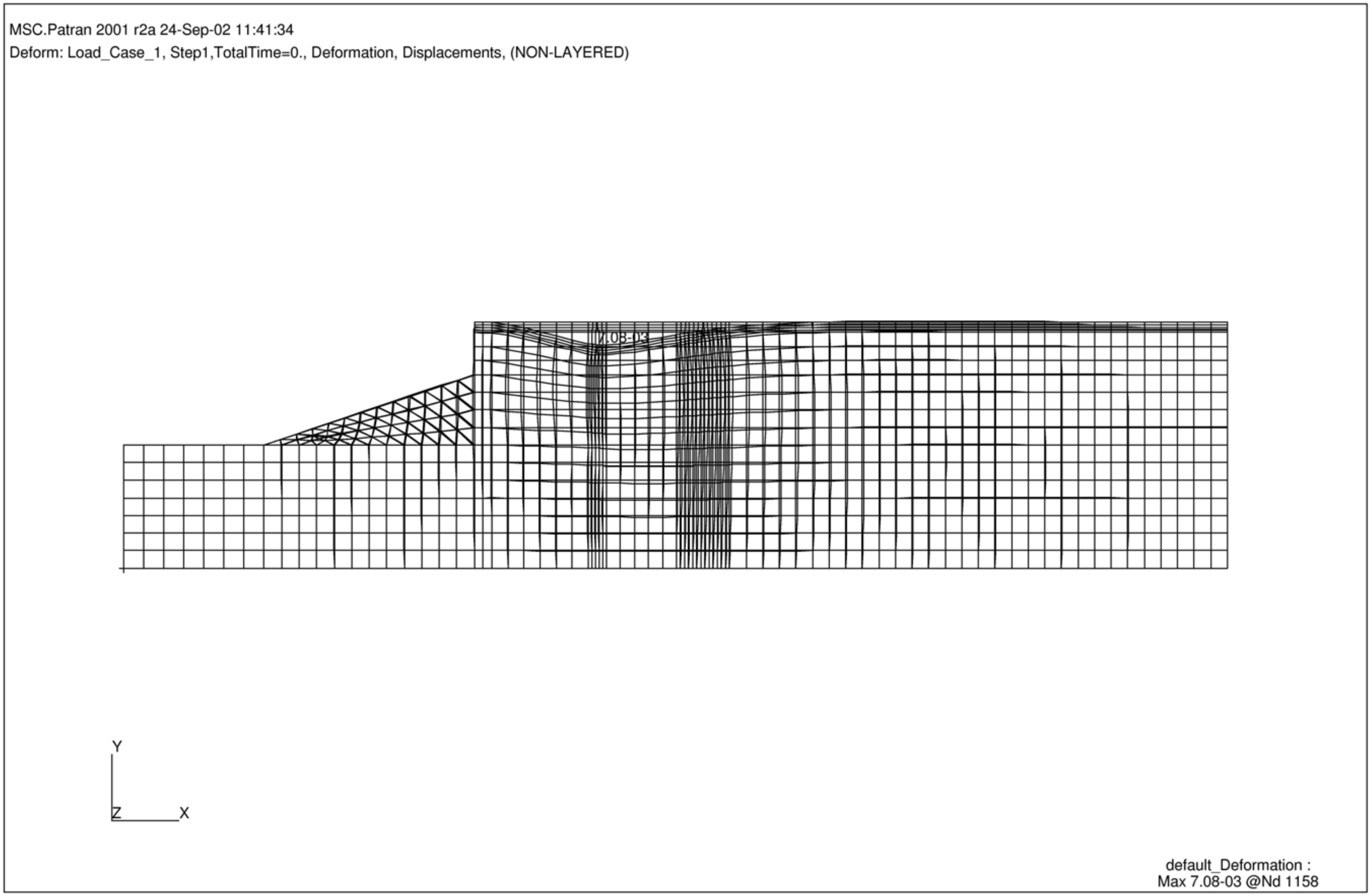


Figure G-25. Load Case 1, 1.0 m Wall, E=2,500 kPa in Zone 3.

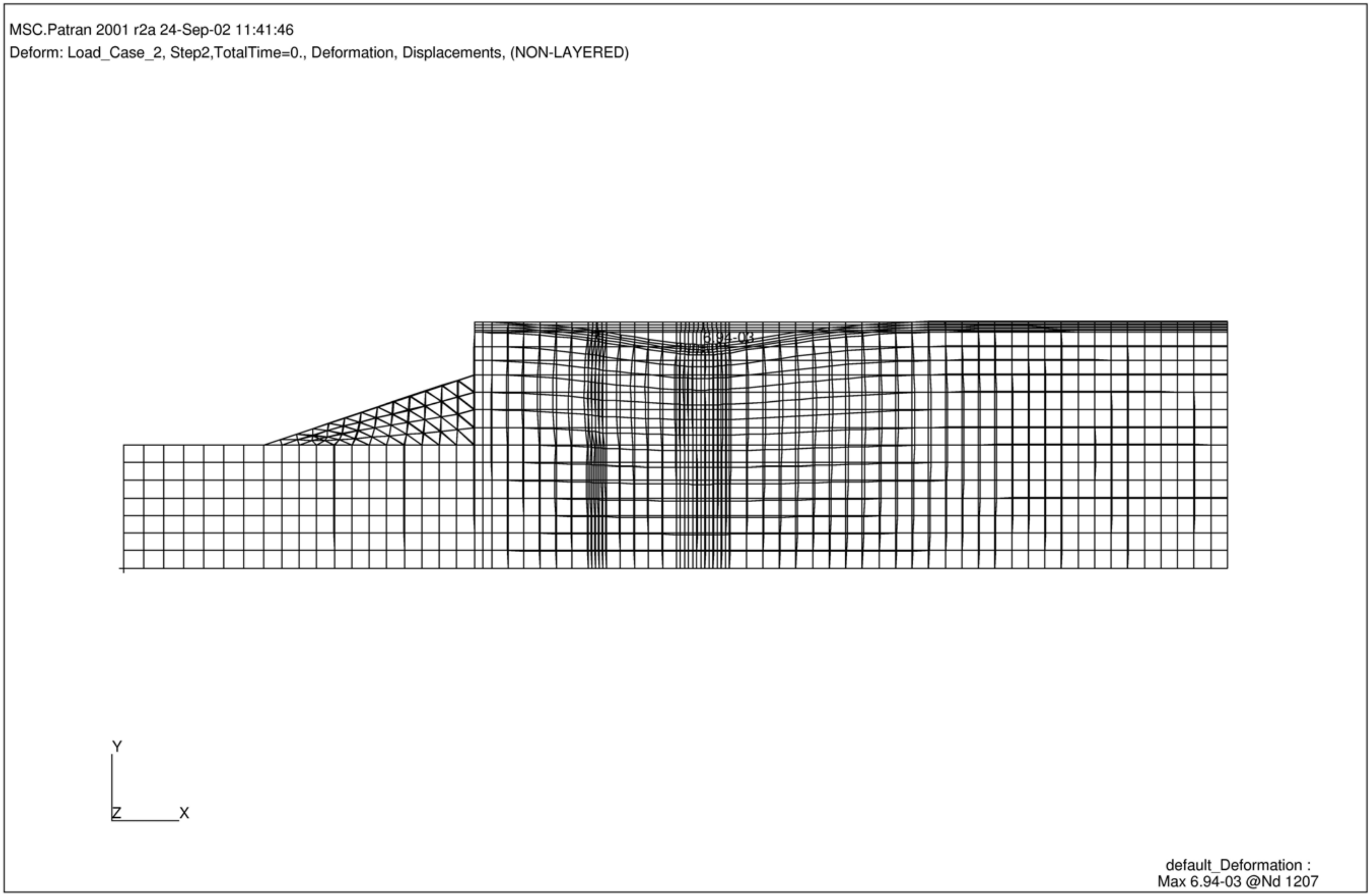


Figure G-26. Load Case 2, 1.0 m Wall, E=2,500 kPa in Zone 3.

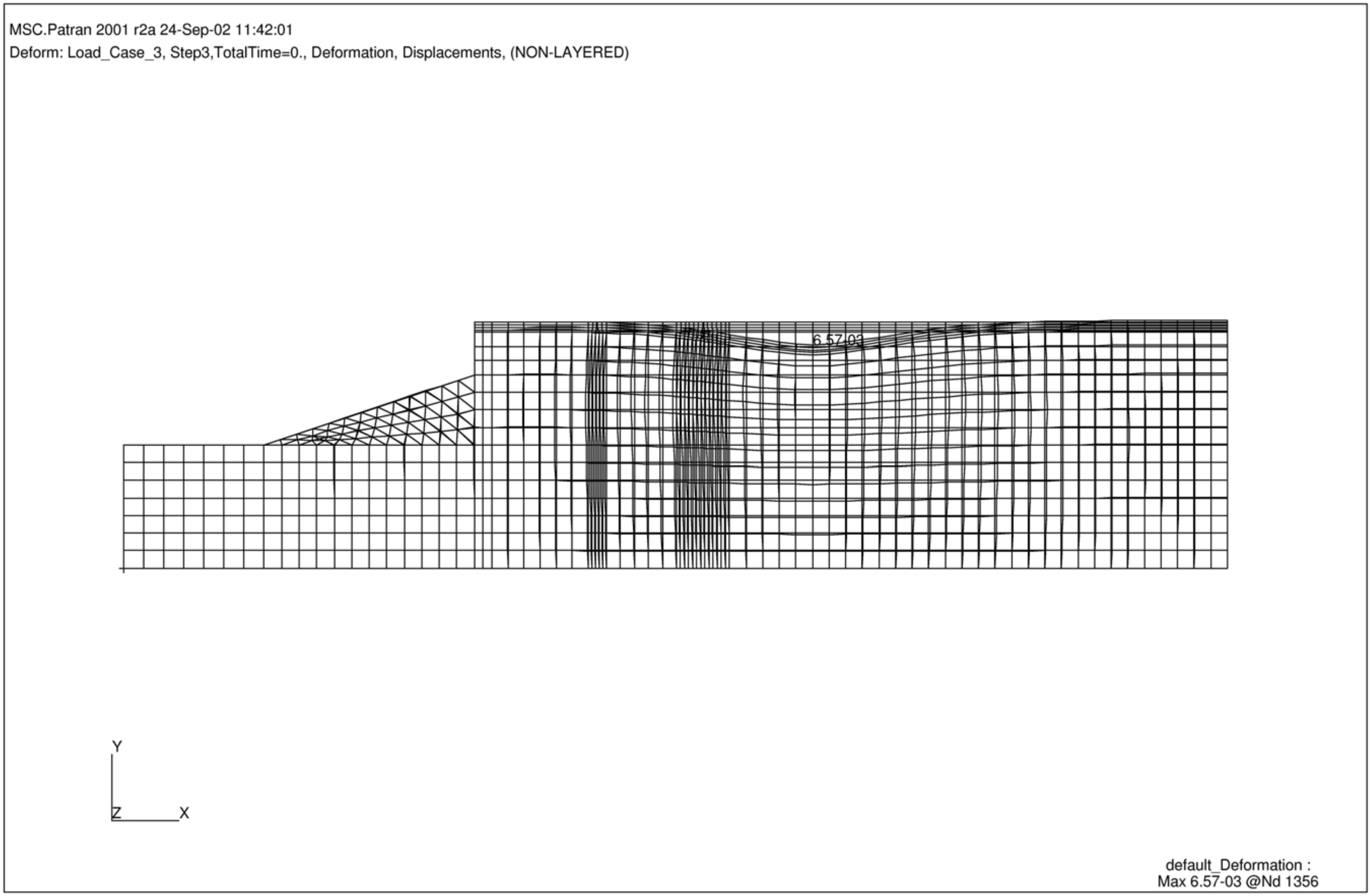


Figure G-27. Load Case 3, 1.0 m Wall, E=2,500 kPa in Zone 3.

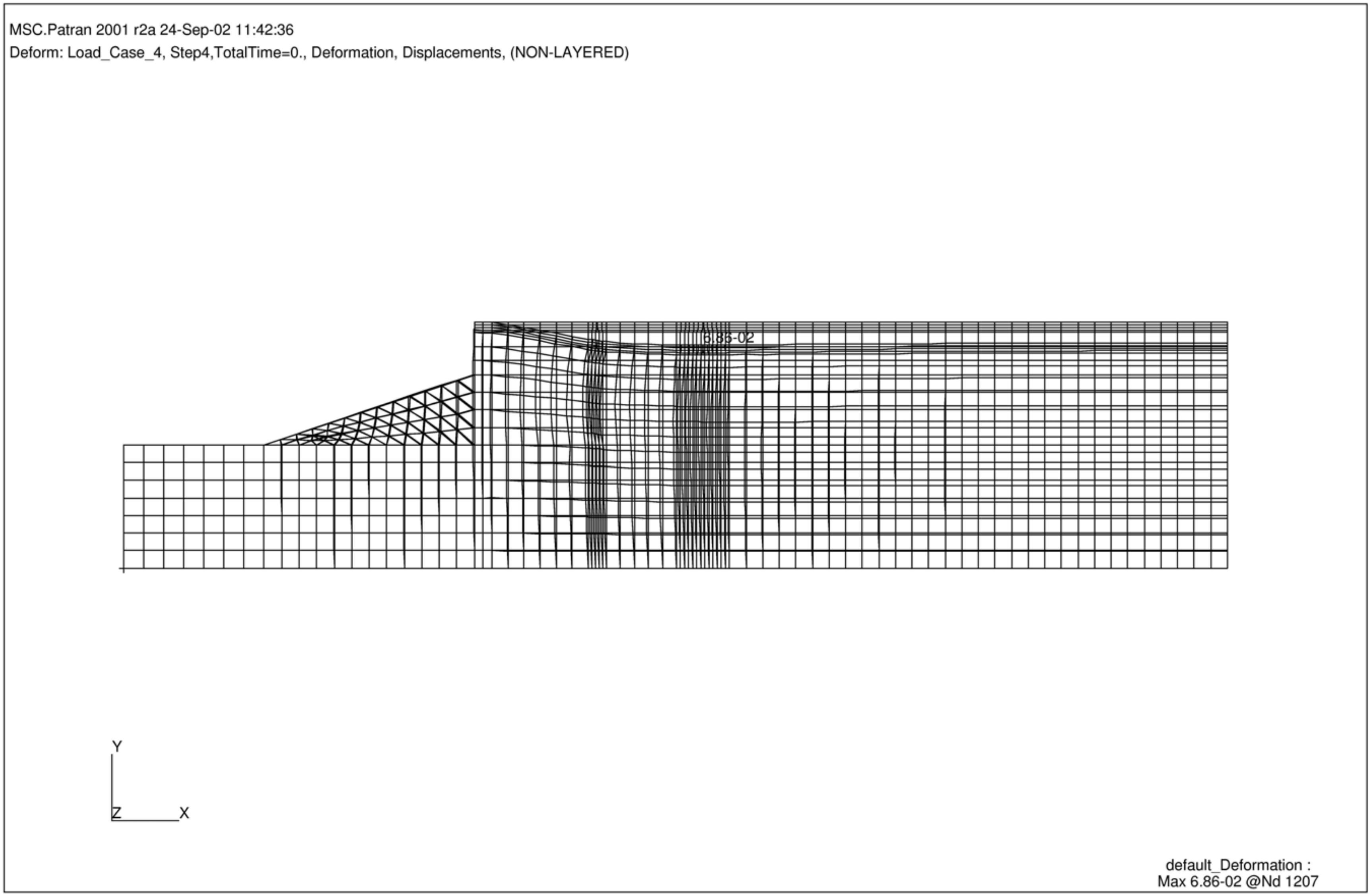


Figure G-28. Load Case 4, 1.0 m Wall, E=2,500 kPa in Zone 3.

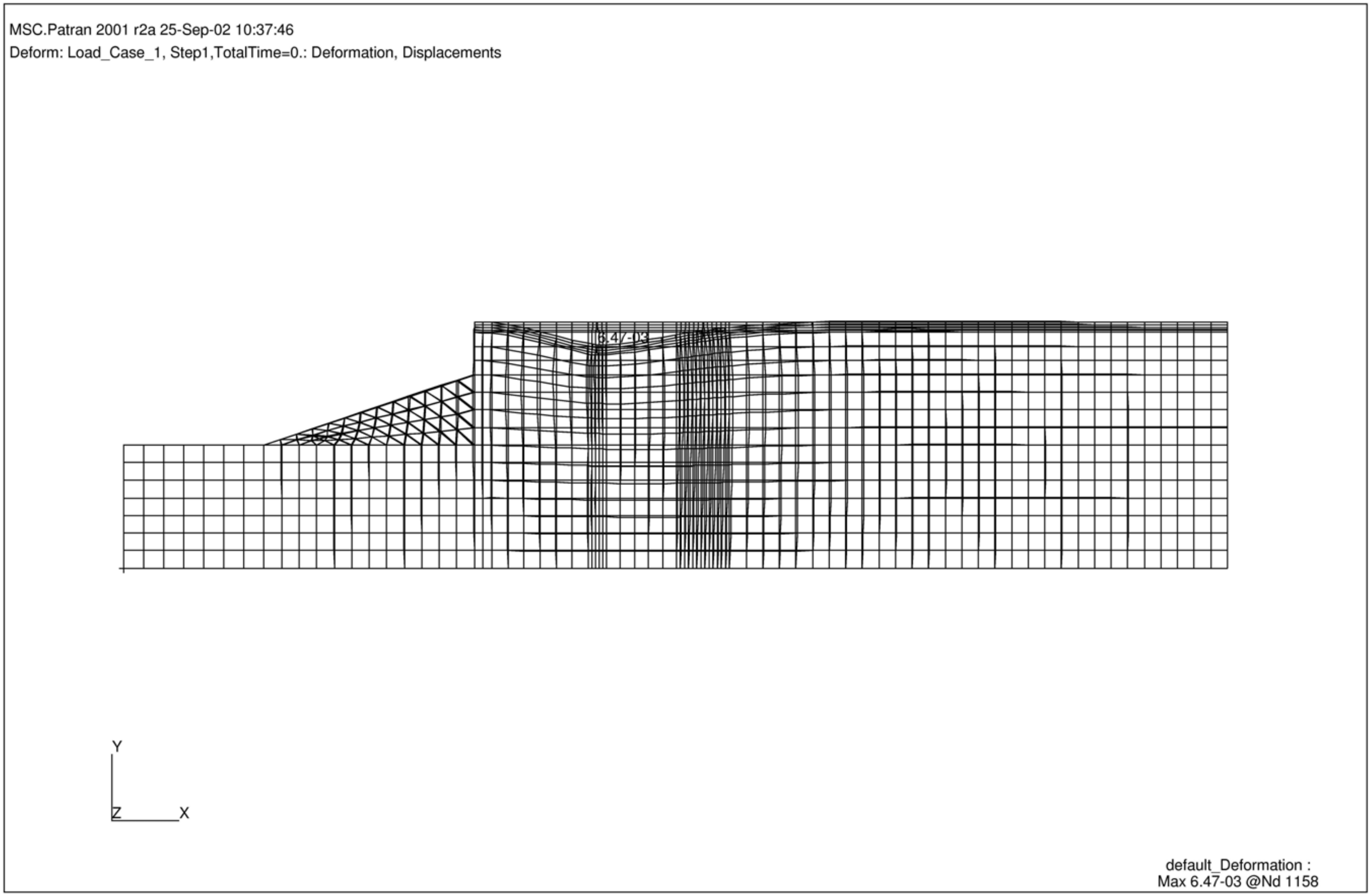


Figure G-29. Load Case 1, 1.0 m Wall, E=5,000 kPa in Zone 3.

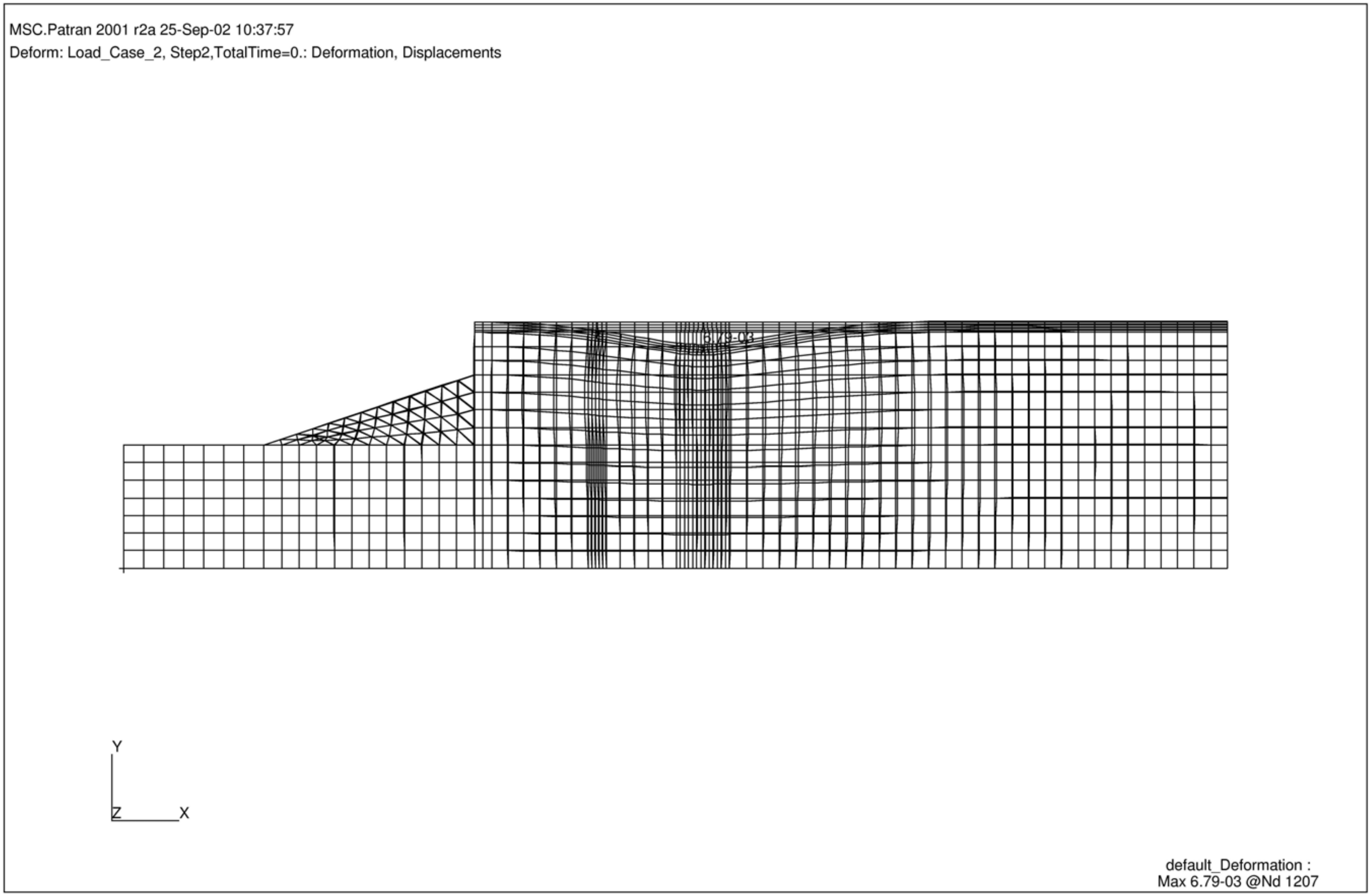


Figure G-30. Load Case 2, 1.0 m Wall, E=5,000 kPa in Zone 3.

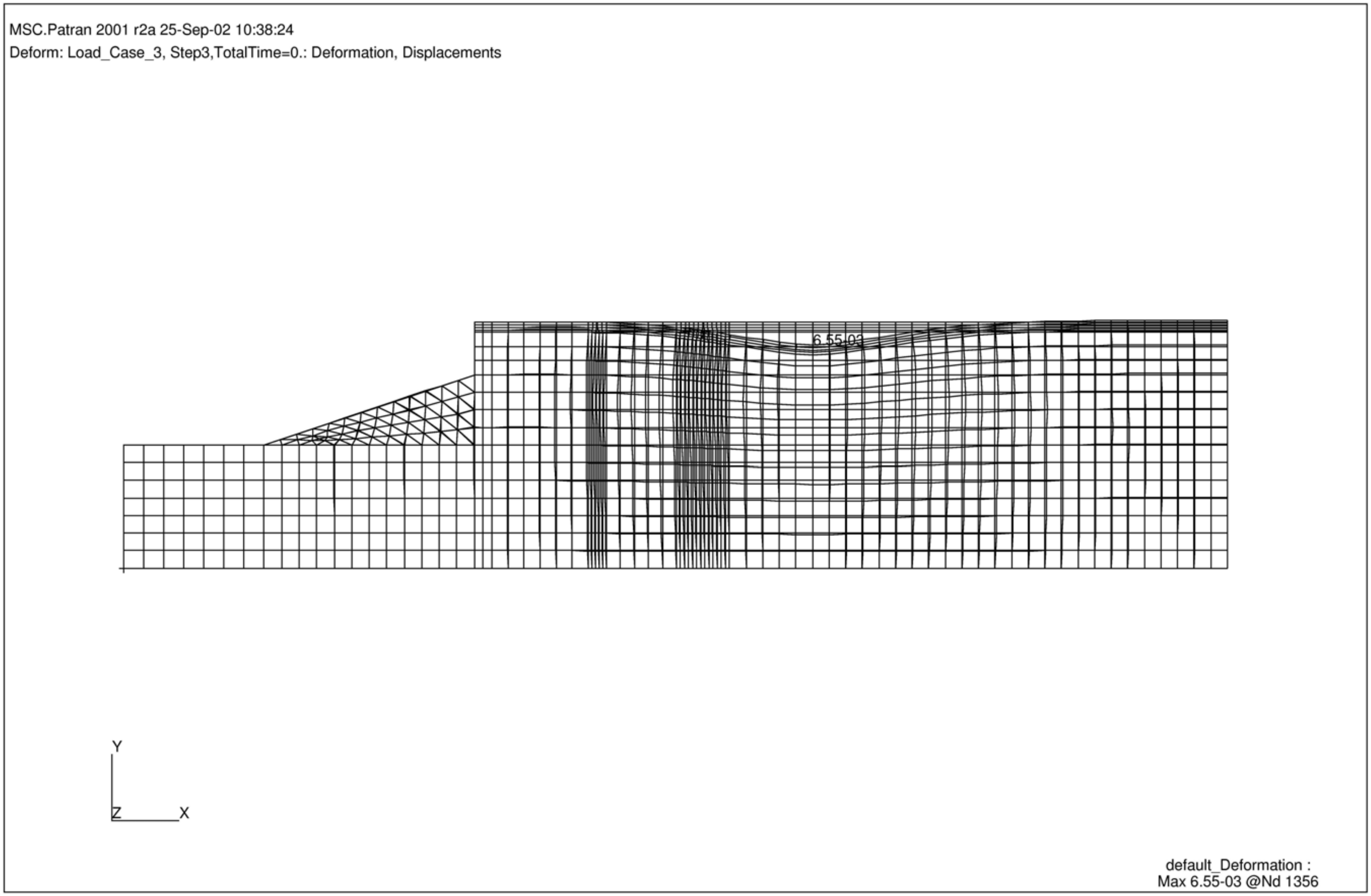


Figure G-31. Load Case 3, 1.0 m Wall, E=5,000 kPa in Zone 3.

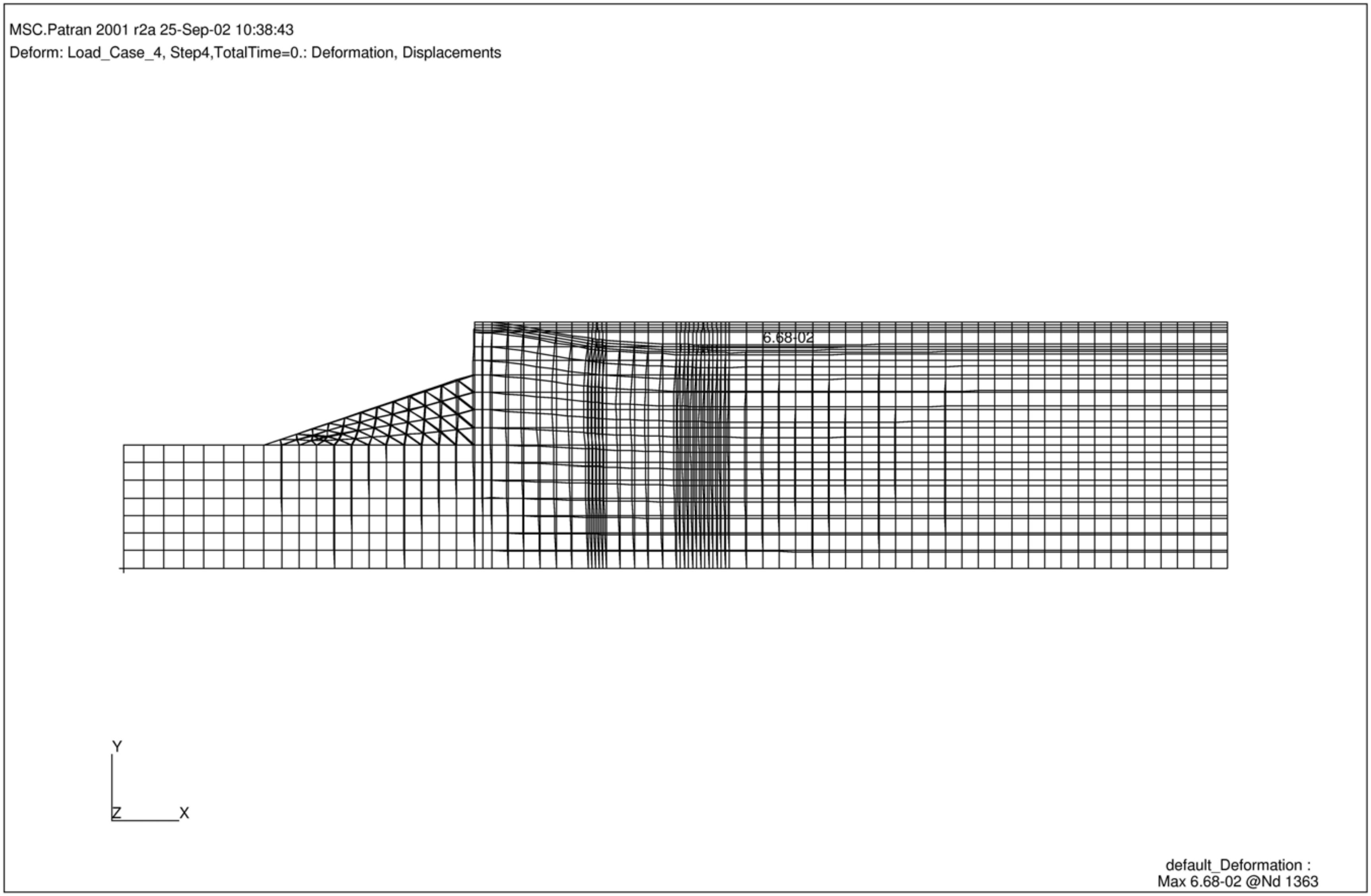


Figure G-32. Load Case 4, 1.0 m Wall, E=5,000 kPa in Zone 3.

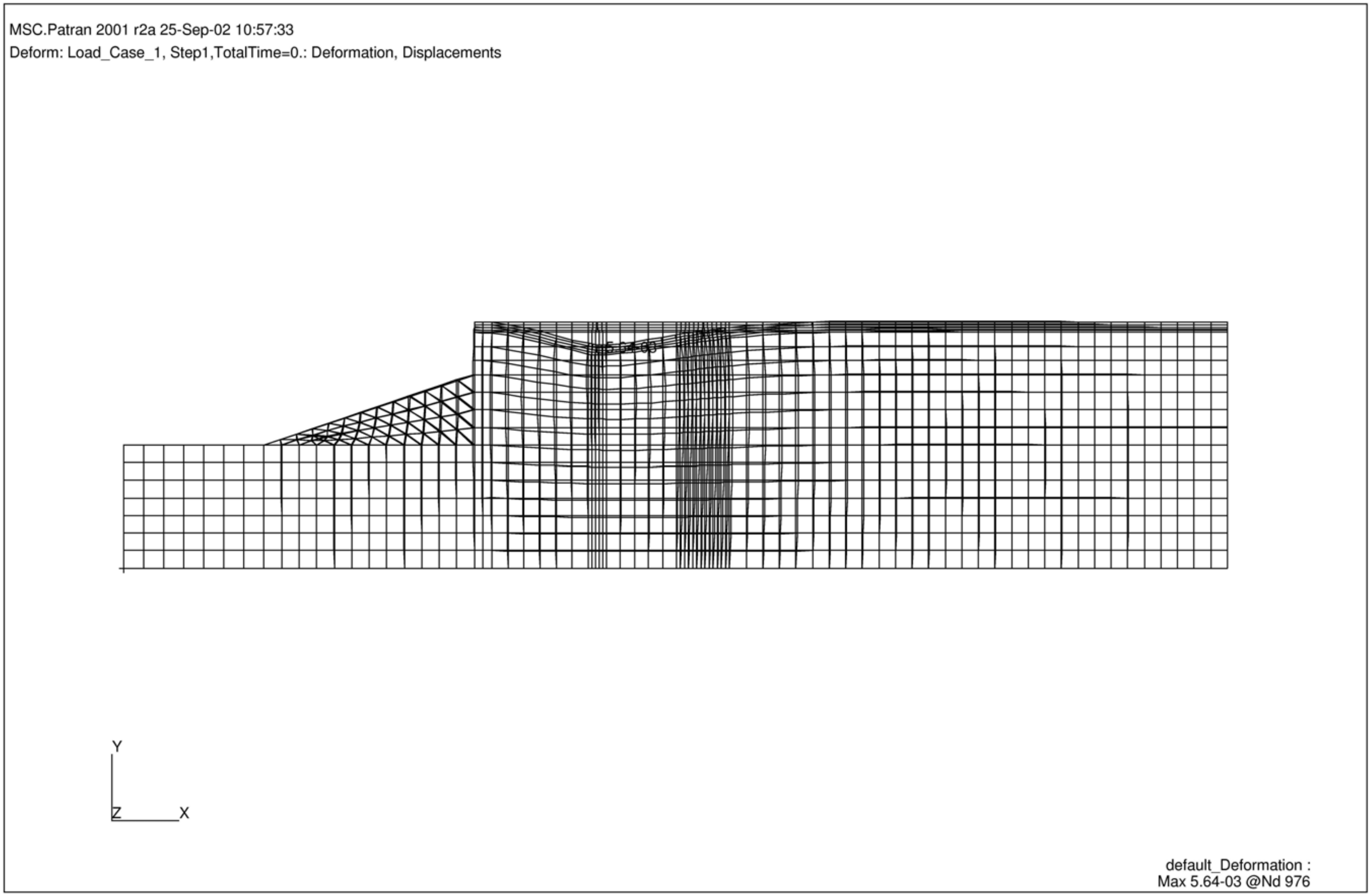


Figure G-33. Load Case 1, 1.0 m Wall, E=10,000 kPa in Zone 3.

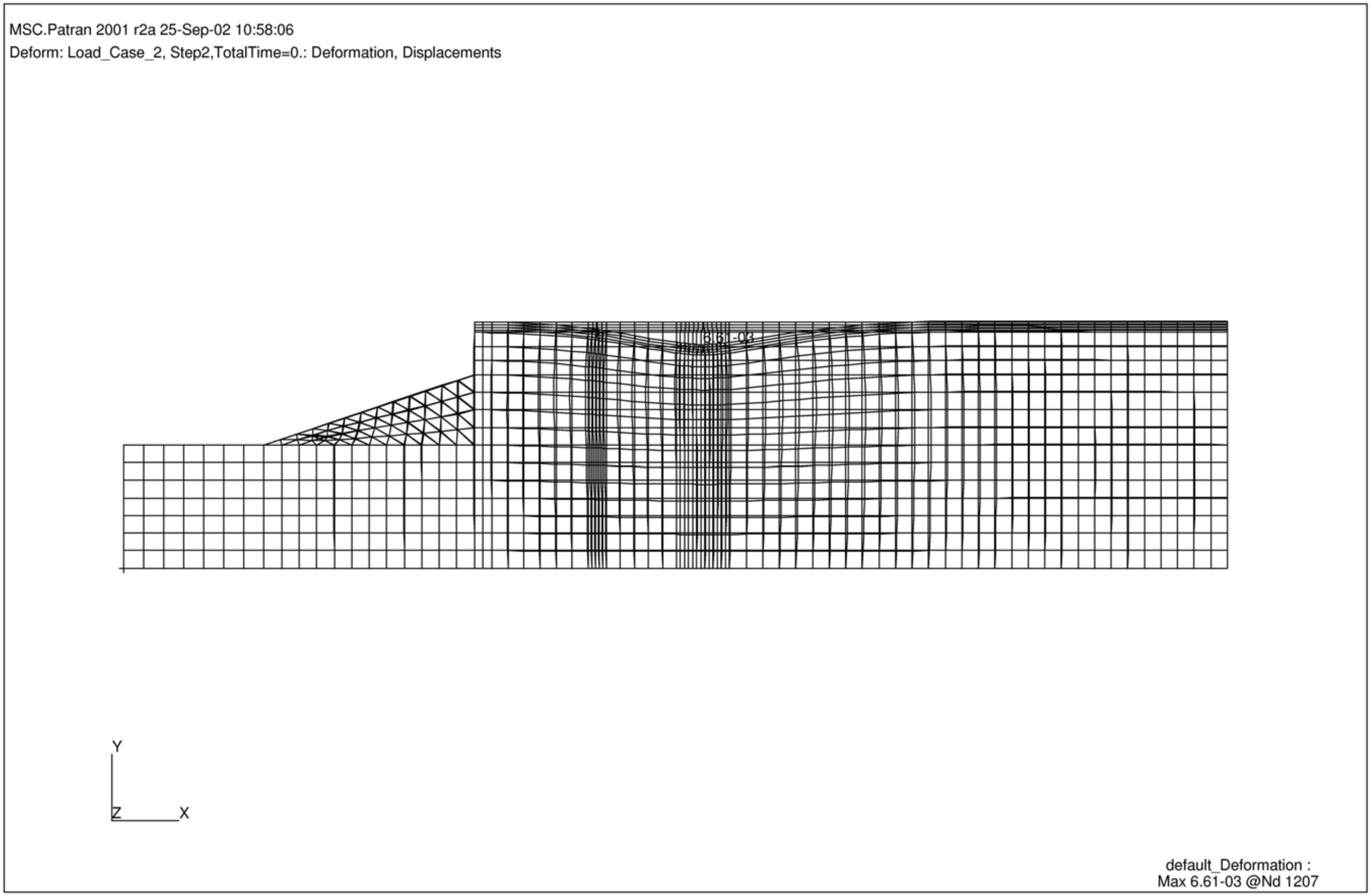


Figure G-34. Load Case 2, 1.0 m Wall, E=10,000 kPa in Zone 3.

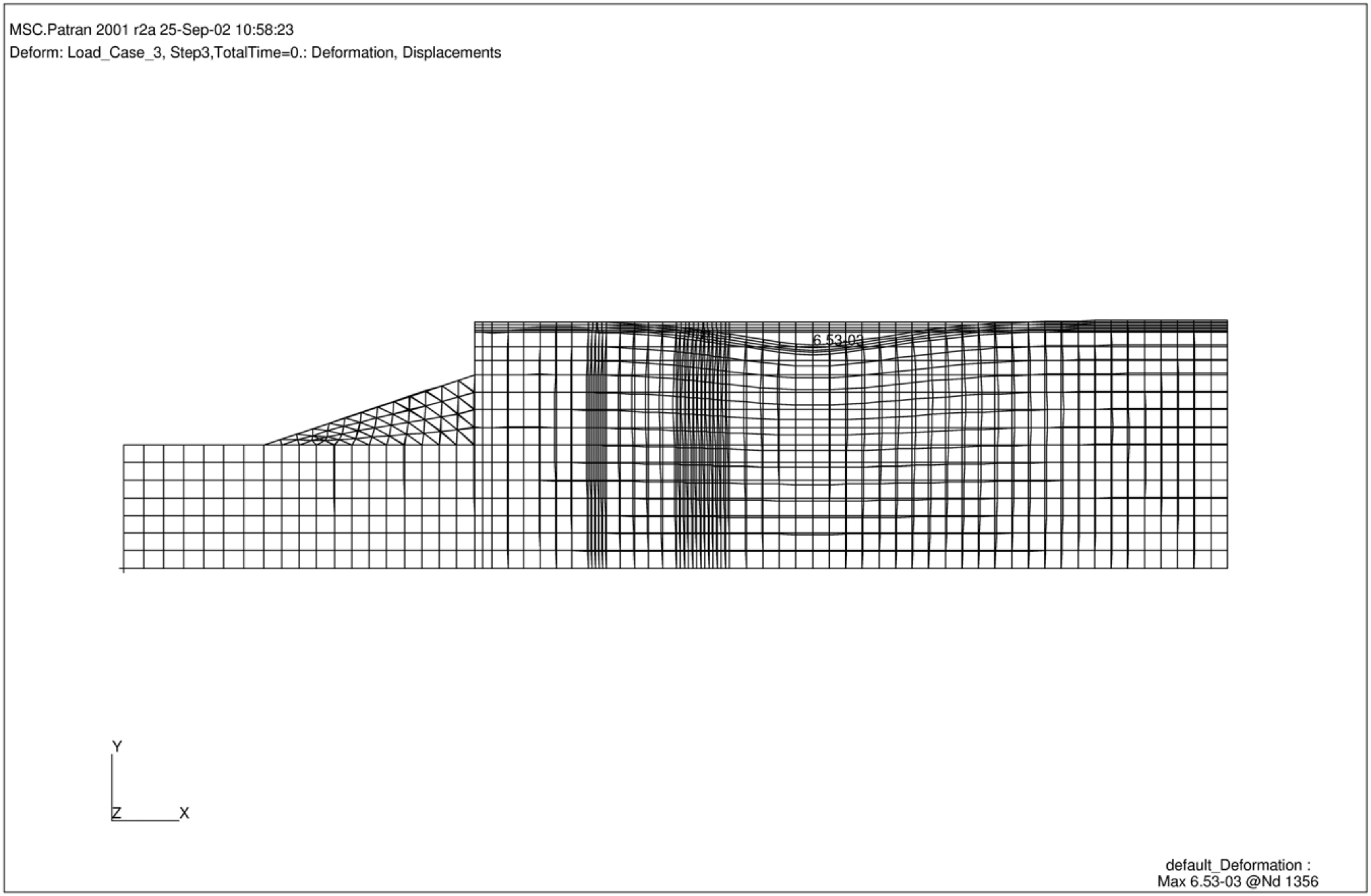


Figure G-35. Load Case 3, 1.0 m Wall, E=10,000 kPa in Zone 3.

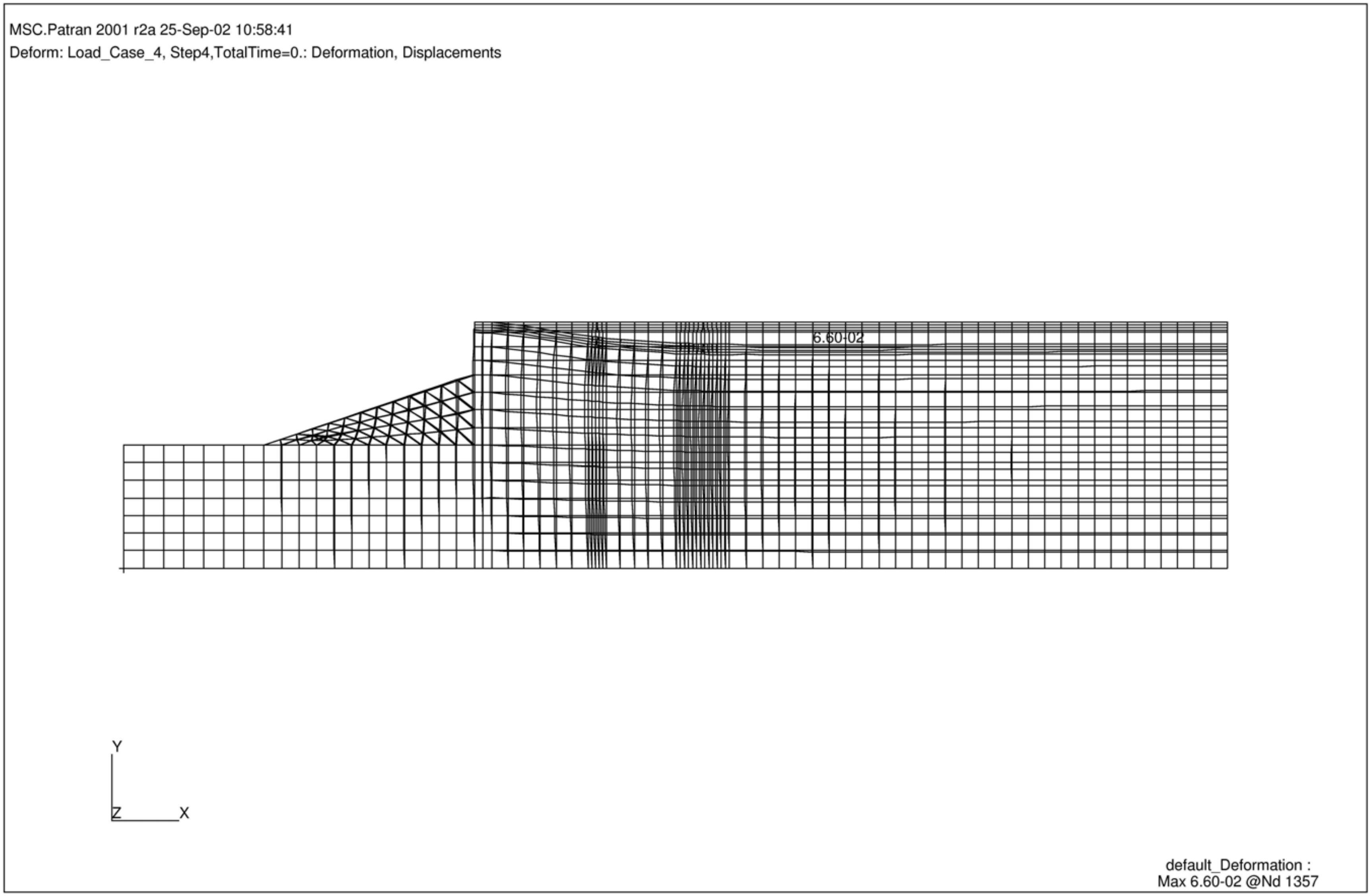


Figure G-36. Load Case 4, 1.0 m Wall, E=10,000 kPa in Zone 3.

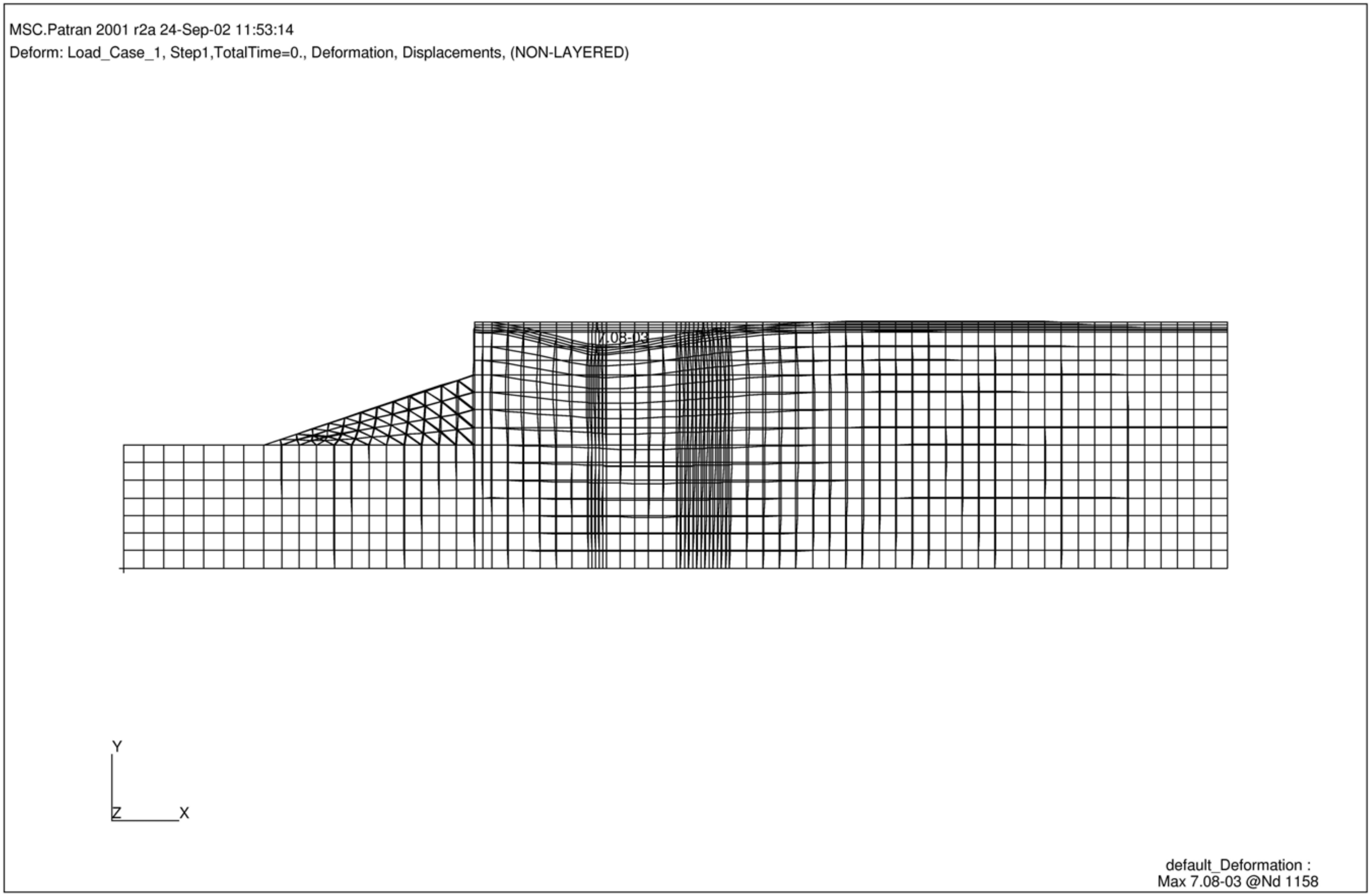


Figure G-37. Load Case 1, 0.5 m Wall , E=5,000 kPa in Zone 3, New Approach Slab.

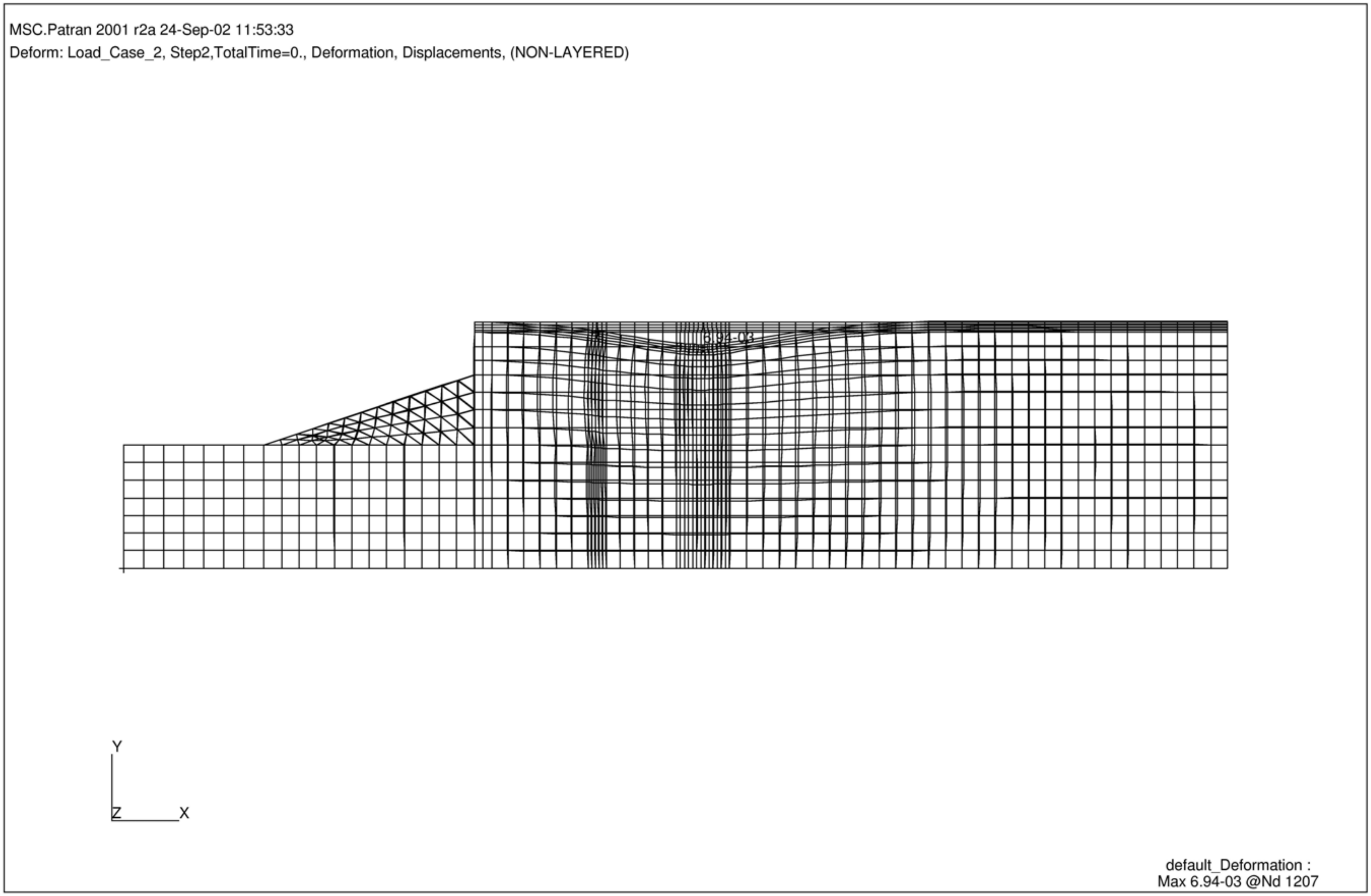


Figure G-38. Load Case 2, 0.5 m Wall , E=5,000 kPa in Zone 3, New Approach Slab.

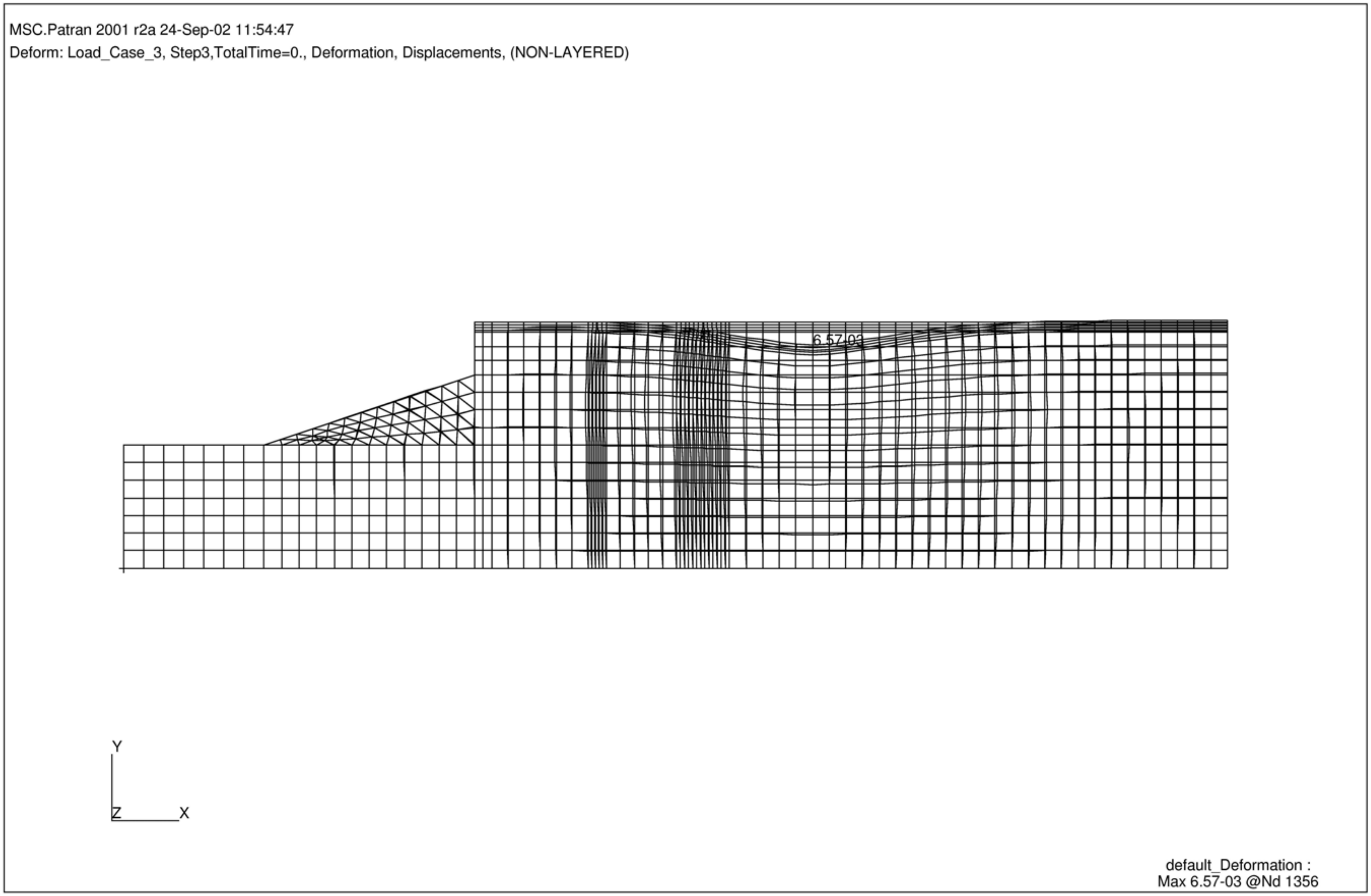


Figure G-39. Load Case 3, 0.5 m Wall , E=5,000 kPa in Zone 3, New Approach Slab.

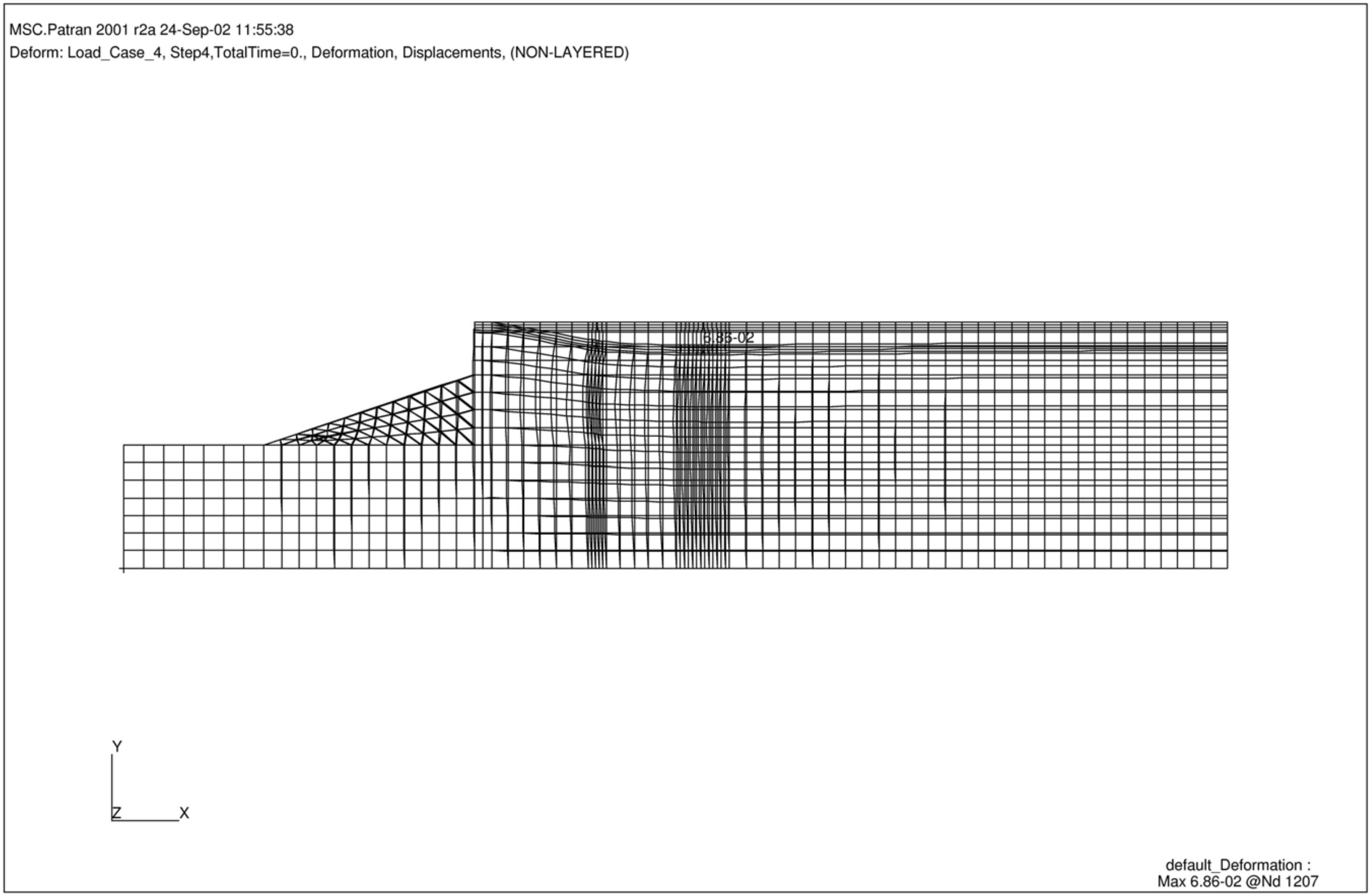


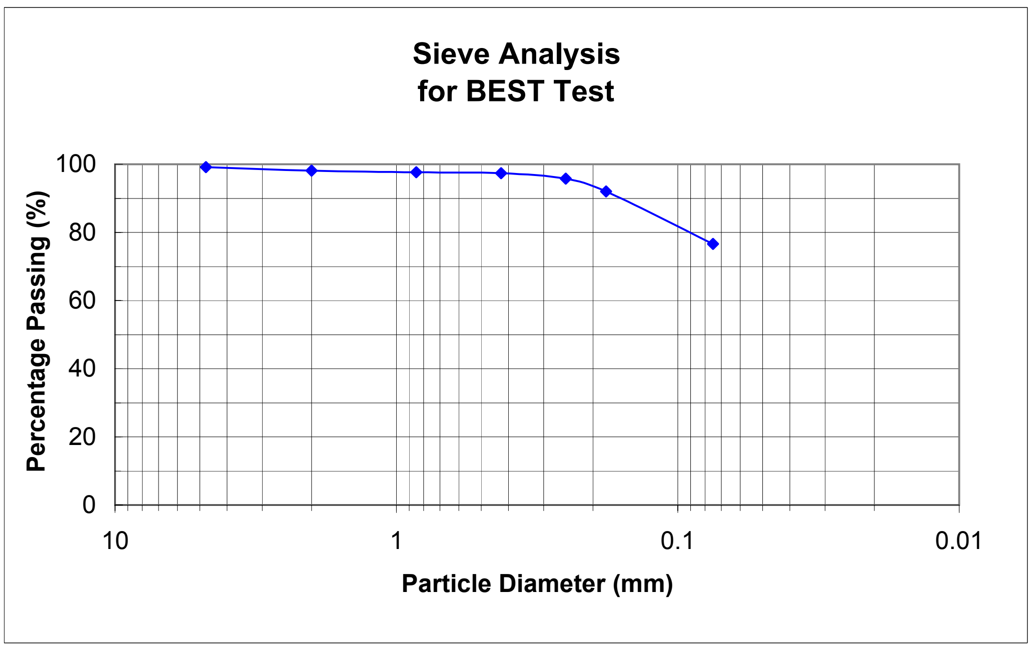
Figure G-40. Load Case 4, 0.5 m Wall , E=5,000 kPa in Zone 3, New Approach Slab.

APPENDIX H BASIC SOIL TESTS

Sieve Analysis for BEST

Sieve Number	Retained (g)	Sieve Mass(g)	Soil Retained(g)	Passing (g)	% Retained	%Passing
4	613.4	608.2	5.2	591.5	0.9	99.1
10	538.5	532.7	5.8	585.7	1.0	98.2
20	371.5	368.7	2.8	582.9	0.5	97.7
40	396.3	394.4	1.9	581.0	0.3	97.4
60	526.3	516.7	9.6	571.4	1.6	95.8
80	547.4	525.0	22.4	549.0	3.8	92.0
200	390.5	298.5	92.0	457.0	15.4	76.6
Pan	258.8	256.5	457.0	0.0	76.6	0.0
Total:	3642.7	3500.7	596.7			

Sieve No.	Opening (mm)
4	4.750
10	2.000
20	0.850
40	0.425
60	0.250
80	0.180
200	0.075
Pan	0

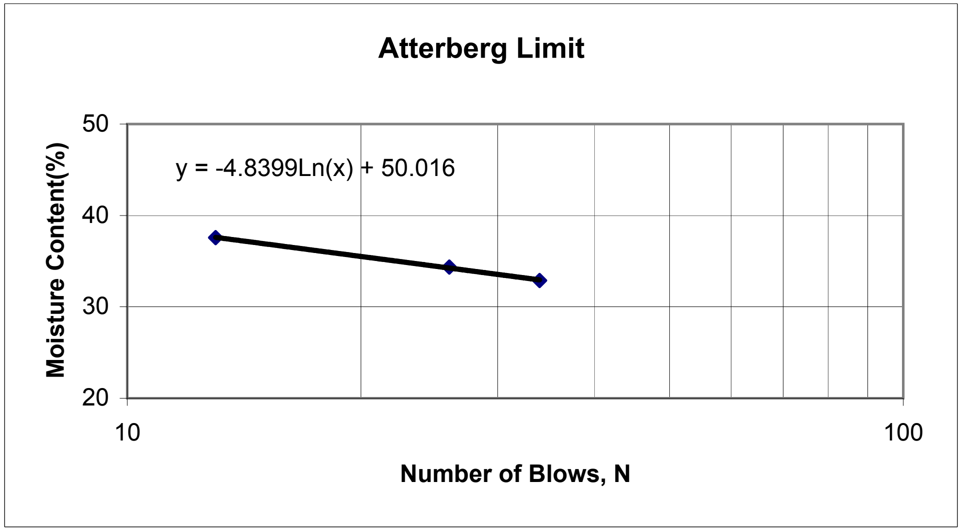


Weight of Tare (g)	368.7
Weight of Tare + Dry Soil - Before Sieving (g)	964.1
Weight of Tare + Dry Soil - After Sieving (g)	509.4
Total Soil (g)	595.4
Soil larger than 0.075 (g)	140.7
Soil finer than 0.075 (g)	454.7

Atterberg Limit Test for Porcelain Clay-2

Liquid Limit

Cup No.	Cup Weight	Cup+Wet Soil	Cup+Dry Soil	Water Wt.	Soil Wt.	Blows	W/C
1	1.00	9.73	7.57	2.16	6.57	34	**32.88**
2	1.00	8.51	6.59	1.92	5.59	26	**34.35**
3	1.00	10.30	7.76	2.54	6.76	13	**37.57**
Liquid Limit							**34.44**



Plastic Limit

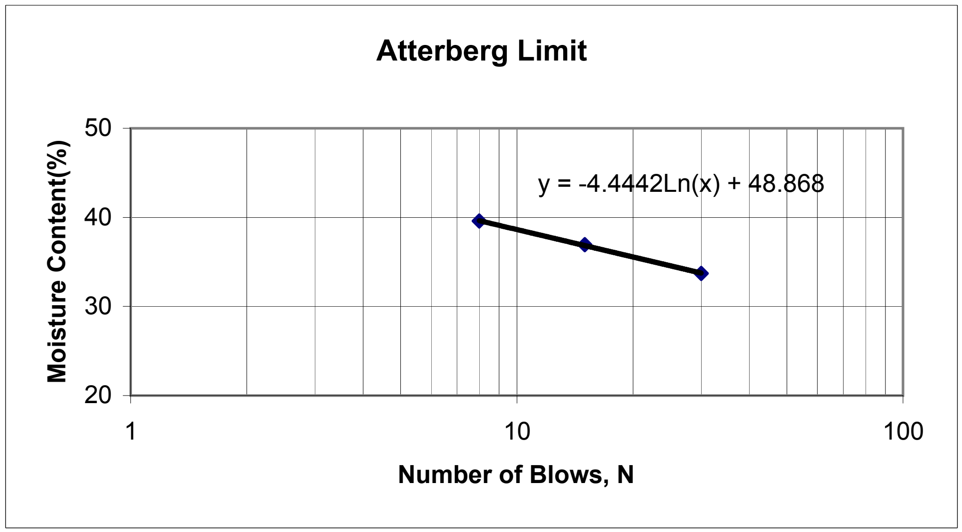
Cup No.	Cup Weight	Cup+Wet Soil	Cup+Dry Soil	Water Wt.	Soil Wt.	Blows	W/C
5	1.00	4.87	4.29	0.58	3.29		**17.63**
6	1.00	4.39	3.85	0.54	2.85		**18.95**
Plastic Limit							**18.29**

Plasticity Index **16.15**

Atterberg Limit Test for Porcelain Clay-3

Liquid Limit

Cup No.	Cup Weight	Cup+Wet Soil	Cup+Dry Soil	Water Wt.	Soil Wt.	Blows	W/C
7	1.00	9.33	7.23	2.1	6.23	30	**33.71**
8	1.00	9.64	7.31	2.33	6.31	15	**36.93**
11	1.00	10.91	8.10	2.81	7.10	8	**39.58**
Liquid Limit							**34.56**



Plastic Limit

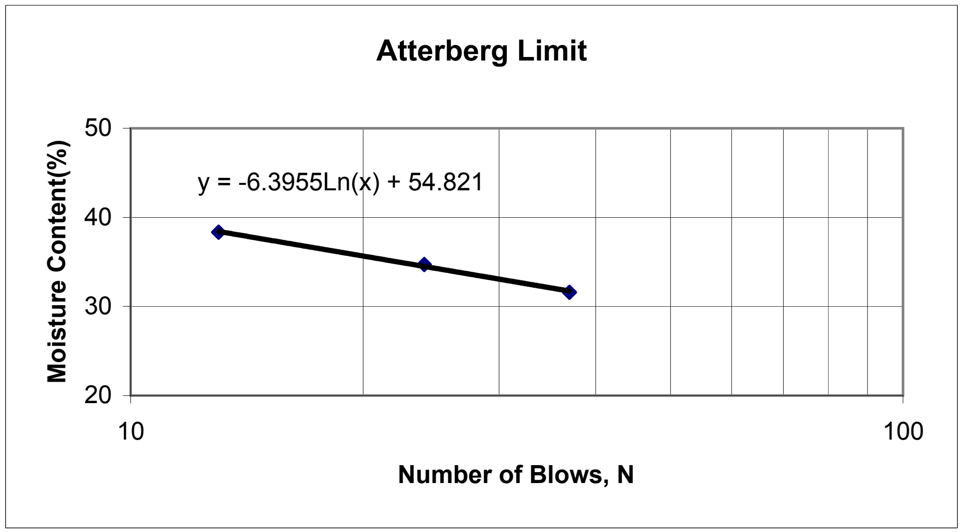
Cup No.	Cup Weight	Cup+Wet Soil	Cup+Dry Soil	Water Wt.	Soil Wt.	Blows	W/C
9	1.00	3.57	3.17	0.40	2.17		**18.43**
10	1.00	4.31	3.79	0.52	2.79		**18.64**
Plastic Limit							**18.54**

Plasticity Index **16.02**

Atterberg Limit Test for Porcelain Clay-4

Liquid Limit

Cup No.	Cup Weight	Cup+Wet Soil	Cup+Dry Soil	Water Wt.	Soil Wt.	Blows	**W/C**
1	1.00	9.58	7.52	2.06	6.52	37	**31.60**
2	1.00	9.42	7.25	2.17	6.25	24	**34.72**
3	1.00	10.24	7.68	2.56	6.68	13	**38.32**
Liquid Limit							**34.23**



Plastic Limit

Cup No.	Cup Weight	Cup+Wet Soil	Cup+Dry Soil	Water Wt.	Soil Wt.	Blows	**W/C**
4	1.00	4.42	3.89	0.53	2.89		**18.34**
5	1.00	4.76	4.19	0.57	3.19		**17.87**
Plastic Limit							**18.10**

Plasticity Index **16.13**

Porcelain Clay

	Length	Height	Volume	Wt of Soil	Tin No.	Wt of Tin	Wt (T+Wet S)	Wt (T+Dry S)	W/C	Dry Unit Wt.	Dry Unit Wt.	Total Unit Wt.	Total Unit Wt.
Sample 1	(mm)	(mm)	(mm^3)	(g)		(g)	(g)	(g)	(%)	(t/m^3)f	(lb/ft^3)	(t/m^3)	(lb/ft^3)
	152.40	254.00	5899343	10889	1	0.98	21.60	17.24	26.8%	1.46	90.87	1.8458	115.23

	Length	Height	Volume	Wt of Soil	Tin No.	Wt of Tin	Wt (T+Wet S)	Wt (T+Dry S)	W/C	Dry Unit Wt.	Dry Unit Wt.	Total Unit Wt.	Total Unit Wt.
Sample 2	(mm)	(mm)	(mm^3)	(g)		(g)	(g)	(g)	(%)	(t/m^3)	(lb/ft^3)	(t/m^3)	(lb/ft^3)
	152.40	254.00	5899343	11519	2	0.97	20.77	16.62	26.5%	1.54	96.35	1.9526	121.90

	Length	Height	Volume	Wt of Soil	Tin No.	Wt of Tin	Wt (T+Wet S)	Wt (T+Dry S)	W/C	Dry Unit Wt.	Dry Unit Wt.	Total Unit Wt.	Total Unit Wt.
Sample 3	(mm)	(mm)	(mm^3)	(g)		(g)	(g)	(g)	(%)	(t/m^3)	(lb/ft^3)	(t/m^3)	(lb/ft^3)
	152.40	254.00	5899343	11684	3	0.97	30.67	24.52	26.1%	1.57	98.04	1.9806	123.65

	Length	Height	Volume	Wt (tin+w soil)	Tin No.	Wt of Tin	Wt (T+Wet S)	Wt (T+Dry S)	W/C	Dry Unit Wt.	Dry Unit Wt.	Total Unit Wt.	Total Unit Wt.
Sample 4	(mm)	(mm)	(mm^3)	(g)		(g)	(g)	(g)	(%)	(t/m^3)	(lb/ft^3)	(t/m^3)	(lb/ft^3)
	152.40	254.00	5899343	11347	4	0.98	20.73	16.60	26.4%	1.52	94.97	1.9234	120.08

Average	26.6%	1.4884	92.92	1.9256	120.22

Sample No. **Porcelain Clay 2**

Depth		ft
Dry Unit Weight	96.35	lb/ft^3
W/C	26.5	%
σ_3	0	psi
Strain rate	0.8	mm/min
Dia. of Sample	1.5	in
L. of Sample	3.15	in
Weight		g
# Dis Trans:	8	y=0.0157x-1.2853
# Force Trans:	CZ0246	y=15.127x-90.406

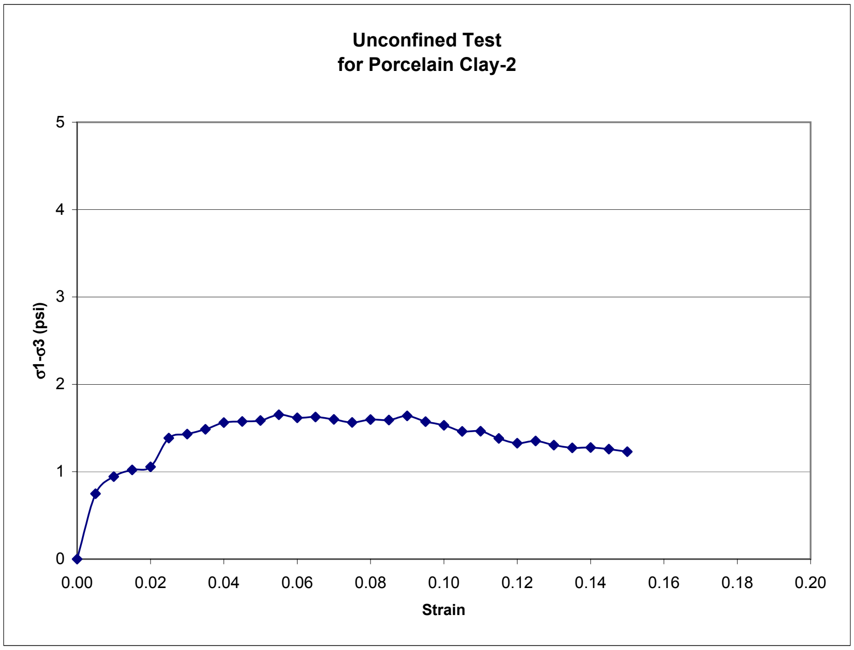
$Cu = q_u/2$ = 0.826 psi

σ_{max} = 1.65 psi

$0.25\sigma_{max}$ = 0.4132 psi

ε_{max} (at $0.25\sigma_{max}$) = 0.0025

E_{25} = 165.3 psi



Time (min)	Disp. (mV)	Force (mV)	Disp. (in)	Load Cell f (lb)	Sig_a*(As-Ap) (lb)	Wp (lb)	Total F (lb)	Corrected A (in^2)	Strain (in/in)	Total Stress (psi)	σ_1-σ_3 (psi)
0	0.000	6.277	0.000	0.00	0.00	0	0.00	1.767	0.000	0.00	0.00
0.5	1.003	6.365	0.016	1.33	0.00	0	1.33	1.780	0.005	0.75	0.75
1	2.006	6.389	0.031	1.69	0.00	0	1.69	1.794	0.010	0.94	0.94
1.5	3.009	6.399	0.047	1.85	0.00	0	1.85	1.808	0.015	1.02	1.02
2	4.012	6.404	0.063	1.92	0.00	0	1.92	1.821	0.020	1.05	1.05
2.5	5.015	6.445	0.079	2.54	0.00	0	2.54	1.835	0.025	1.38	1.38
3	6.018	6.452	0.094	2.65	0.00	0	2.65	1.849	0.030	1.43	1.43
3.5	7.021	6.46	0.110	2.77	0.00	0	2.77	1.863	0.035	1.49	1.49
4	8.024	6.471	0.126	2.93	0.00	0	2.93	1.878	0.040	1.56	1.56
4.5	9.028	6.474	0.142	2.98	0.00	0	2.98	1.892	0.045	1.57	1.57
5	10.031	6.477	0.157	3.03	0.00	0	3.03	1.907	0.050	1.59	1.59
5.5	11.034	6.487	0.173	3.18	0.00	0	3.18	1.922	0.055	**1.65**	**1.65**
6	12.037	6.484	0.189	3.13	0.00	0	3.13	1.937	0.060	1.62	1.62
6.5	13.040	6.487	0.205	3.18	0.00	0	3.18	1.952	0.065	1.63	1.63
7	14.043	6.485	0.220	3.15	0.00	0	3.15	1.968	0.070	1.60	1.60
7.5	15.046	6.482	0.236	3.10	0.00	0	3.10	1.983	0.075	1.56	1.56
8	16.049	6.488	0.252	3.19	0.00	0	3.19	1.999	0.080	1.60	1.60
8.5	17.052	6.489	0.268	3.21	0.00	0	3.21	2.015	0.085	1.59	1.59
9	18.055	6.497	0.283	3.33	0.00	0	3.33	2.031	0.090	1.64	1.64
9.5	19.058	6.49	0.299	3.22	0.00	0	3.22	2.047	0.095	1.57	1.57
10	20.061	6.486	0.315	3.16	0.00	0	3.16	2.064	0.100	1.53	1.53
10.5	21.064	6.478	0.331	3.04	0.00	0	3.04	2.081	0.105	1.46	1.46
11	22.067	6.48	0.346	3.07	0.00	0	3.07	2.098	0.110	1.46	1.46
11.5	23.070	6.47	0.362	2.92	0.00	0	2.92	2.115	0.115	1.38	1.38
12	24.073	6.464	0.378	2.83	0.00	0	2.83	2.132	0.120	1.33	1.33
12.5	25.076	6.469	0.394	2.90	0.00	0	2.90	2.149	0.125	1.35	1.35
13	26.080	6.464	0.409	2.83	0.00	0	2.83	2.167	0.130	1.31	1.31
13.5	27.083	6.461	0.425	2.78	0.00	0	2.78	2.185	0.135	1.27	1.27
14	28.086	6.463	0.441	2.81	0.00	0	2.81	2.203	0.140	1.28	1.28
14.5	29.089	6.462	0.457	2.80	0.00	0	2.80	2.222	0.145	1.26	1.26
15	30.092	6.459	0.472	2.75	0.00	0	2.75	2.240	0.150	1.23	1.23

Sample No. **Porcelain Clay 3**

Depth		ft
Dry Unit Weight	98.04	lb/ft^3
W/C	26.1	%
σ_3	0	psi
Strain Rate	0.8	mm/min
Dia. of Sample	1.5	in
L. of Sample	3.15	in
Weight		g
# Dis Trans:	8	y=0.0157x-1.2853
# Force Trans:	CZ0246	y=15.127x-90.406

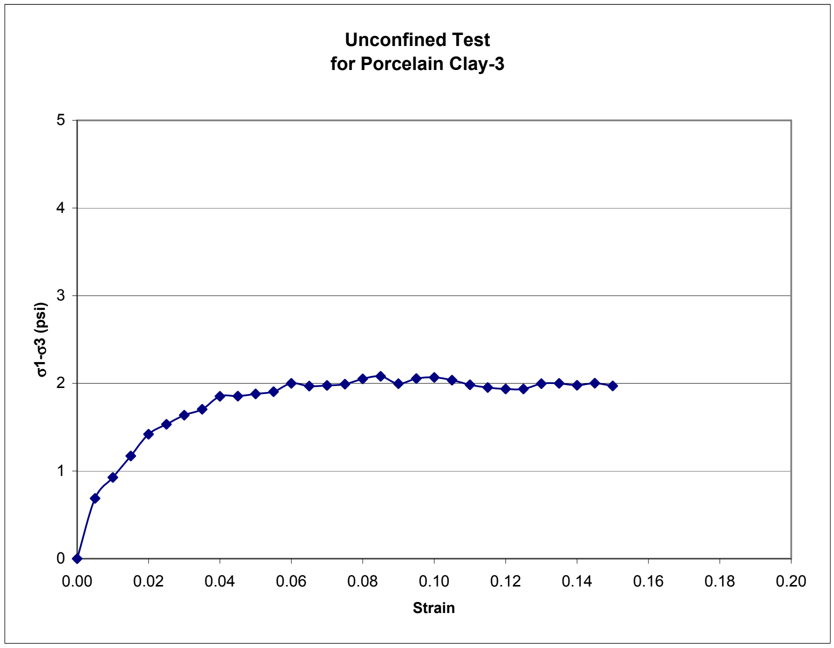
$Cu = q_u/2$ = 0.952 psi

σ_{max} = 2.08 psi

$0.25\sigma_{max}$ = 0.5199 psi

ε_{max} (at $0.25\sigma_{max}$) = 0.00355

E_{25} = 146.4 psi



Time (min)	Disp. (mV)	Force (mV)	Disp. (in)	Load Cell f (lb)	Sig_a*(As-Ap) (lb)	Wp (lb)	Total F (lb)	Corrected A (in^2)	Strain (in/in)	Total Stress (psi)	$\sigma_1-\sigma_3$ (psi)
0	0.000	6.266	0.000	0.00	0.00	0	0.00	1.767	0.000	0.00	0.00
0.5	1.003	6.347	0.016	1.23	0.00	0	1.23	1.780	0.005	0.69	0.69
1	2.006	6.376	0.031	1.66	0.00	0	1.66	1.794	0.010	0.93	0.93
1.5	3.009	6.406	0.047	2.12	0.00	0	2.12	1.808	0.015	1.17	1.17
2	4.012	6.437	0.063	2.59	0.00	0	2.59	1.821	0.020	1.42	1.42
2.5	5.015	6.452	0.079	2.81	0.00	0	2.81	1.835	0.025	1.53	1.53
3	6.018	6.466	0.094	3.03	0.00	0	3.03	1.849	0.030	1.64	1.64
3.5	7.021	6.476	0.110	3.18	0.00	0	3.18	1.863	0.035	1.70	1.70
4	8.024	6.496	0.126	3.48	0.00	0	3.48	1.878	0.040	1.85	1.85
4.5	9.028	6.498	0.142	3.51	0.00	0	3.51	1.892	0.045	1.85	1.85
5	10.031	6.503	0.157	3.59	0.00	0	3.59	1.907	0.050	1.88	1.88
5.5	11.034	6.508	0.173	3.66	0.00	0	3.66	1.922	0.055	1.90	1.90
6	12.037	6.522	0.189	3.87	0.00	0	3.87	1.937	0.060	2.00	2.00
6.5	13.040	6.52	0.205	3.84	0.00	0	3.84	1.952	0.065	1.97	1.97
7	14.043	6.523	0.220	3.89	0.00	0	3.89	1.968	0.070	1.98	1.98
7.5	15.046	6.527	0.236	3.95	0.00	0	3.95	1.983	0.075	1.99	1.99
8	16.049	6.537	0.252	4.10	0.00	0	4.10	1.999	0.080	2.05	2.05
8.5	17.052	6.543	0.268	4.19	0.00	0	4.19	2.015	0.085	**2.08**	**2.08**
9	18.055	6.534	0.283	4.05	0.00	0	4.05	2.031	0.090	2.00	2.00
9.5	19.058	6.544	0.299	4.21	0.00	0	4.21	2.047	0.095	2.05	2.05
10	20.061	6.548	0.315	4.27	0.00	0	4.27	2.064	0.100	2.07	2.07
10.5	21.064	6.546	0.331	4.24	0.00	0	4.24	2.081	0.105	2.04	2.04
11	22.067	6.541	0.346	4.16	0.00	0	4.16	2.098	0.110	1.98	1.98
11.5	23.070	6.539	0.362	4.13	0.00	0	4.13	2.115	0.115	1.95	1.95
12	24.073	6.539	0.378	4.13	0.00	0	4.13	2.132	0.120	1.94	1.94
12.5	25.076	6.541	0.394	4.16	0.00	0	4.16	2.149	0.125	1.94	1.94
13	26.080	6.552	0.409	4.33	0.00	0	4.33	2.167	0.130	2.00	2.00
13.5	27.083	6.555	0.425	4.37	0.00	0	4.37	2.185	0.135	2.00	2.00
14	28.086	6.554	0.441	4.36	0.00	0	4.36	2.203	0.140	1.98	1.98
14.5	29.089	6.56	0.457	4.45	0.00	0	4.45	2.222	0.145	2.00	2.00
15	30.092	6.558	0.472	4.42	0.00	0	4.42	2.240	0.150	1.97	1.97

Sample No. **Porcelain Clay 4**

Depth		ft
Dry Unit Weight	98.04	lb/ft^3
W/C	26.1	%
σ_3	0	psi
Strain Rate	0.8	mm/min
Dia. of Sample	1.5	in
L. of Sample	3.15	in
Weight		g
# Dis Trans:	8	y=0.0157x-1.2853
# Force Trans:	CZ0246	y=15.127x-90.406

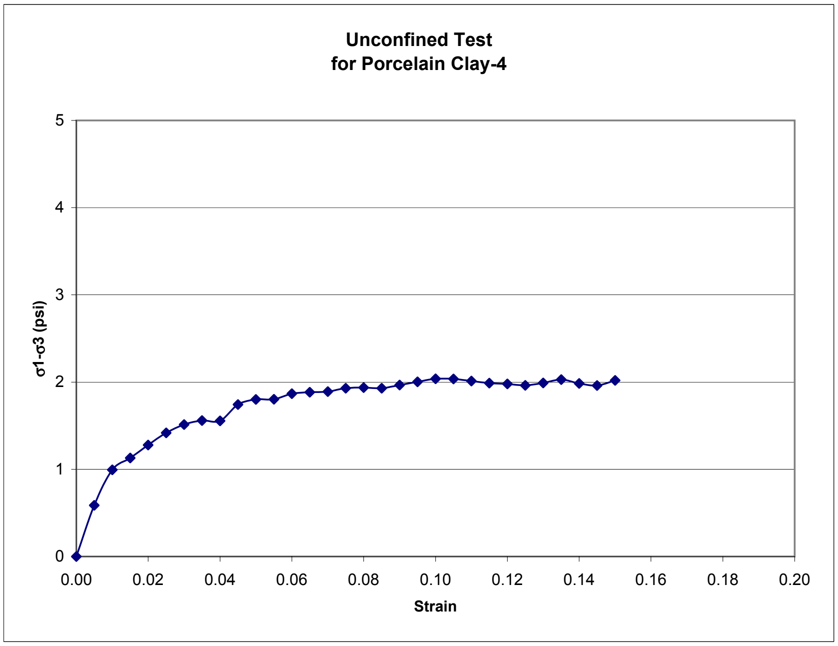
$C_u = q_u/2$ = 0.901 psi

σ_{max} = 2.04 psi

$0.25\sigma_{max}$ = 0.50939 psi

ε_{max} (at $0.25\sigma_{max}$) = 0.0043

E_{25} = 118.5 psi



Time (min)	Disp. (mV)	Force (mV)	Disp. (in)	Load Cell f (lb)	Sig_a*(As-Ap) (lb)	Wp (lb)	Total F (lb)	Corrected A (in^2)	Strain (in/in)	Total Stress (psi)	σ_1-σ_3 (psi)
0	0.000	6.268	0.000	0.00	0.00	0	0.00	1.767	0.000	0.00	0.00
0.5	1.003	6.337	0.016	1.04	0.00	0	1.04	1.780	0.005	0.59	0.59
1	2.006	6.386	0.031	1.78	0.00	0	1.78	1.794	0.010	1.00	1.00
1.5	3.009	6.403	0.047	2.04	0.00	0	2.04	1.808	0.015	1.13	1.13
2	4.012	6.422	0.063	2.33	0.00	0	2.33	1.821	0.020	1.28	1.28
2.5	5.015	6.44	0.079	2.60	0.00	0	2.60	1.835	0.025	1.42	1.42
3	6.018	6.453	0.094	2.80	0.00	0	2.80	1.849	0.030	1.51	1.51
3.5	7.021	6.46	0.110	2.90	0.00	0	2.90	1.863	0.035	1.56	1.56
4	8.024	6.461	0.126	2.92	0.00	0	2.92	1.878	0.040	1.55	1.55
4.5	9.028	6.486	0.142	3.30	0.00	0	3.30	1.892	0.045	1.74	1.74
5	10.031	6.495	0.157	3.43	0.00	0	3.43	1.907	0.050	1.80	1.80
5.5	11.034	6.497	0.173	3.46	0.00	0	3.46	1.922	0.055	1.80	1.80
6	12.037	6.507	0.189	3.62	0.00	0	3.62	1.937	0.060	1.87	1.87
6.5	13.040	6.511	0.205	3.68	0.00	0	3.68	1.952	0.065	1.88	1.88
7	14.043	6.514	0.220	3.72	0.00	0	3.72	1.968	0.070	1.89	1.89
7.5	15.046	6.521	0.236	3.83	0.00	0	3.83	1.983	0.075	1.93	1.93
8	16.049	6.524	0.252	3.87	0.00	0	3.87	1.999	0.080	1.94	1.94
8.5	17.052	6.525	0.268	3.89	0.00	0	3.89	2.015	0.085	1.93	1.93
9	18.055	6.532	0.283	3.99	0.00	0	3.99	2.031	0.090	1.97	1.97
9.5	19.058	6.539	0.299	4.10	0.00	0	4.10	2.047	0.095	2.00	2.00
10	20.061	6.546	0.315	4.21	0.00	0	4.21	2.064	0.100	**2.04**	**2.04**
10.5	21.064	6.548	0.331	4.24	0.00	0	4.24	2.081	0.105	2.04	2.04
11	22.067	6.547	0.346	4.22	0.00	0	4.22	2.098	0.110	2.01	2.01
11.5	23.070	6.546	0.362	4.21	0.00	0	4.21	2.115	0.115	1.99	1.99
12	24.073	6.547	0.378	4.22	0.00	0	4.22	2.132	0.120	1.98	1.98
12.5	25.076	6.547	0.394	4.22	0.00	0	4.22	2.149	0.125	1.96	1.96
13	26.080	6.553	0.409	4.31	0.00	0	4.31	2.167	0.130	1.99	1.99
13.5	27.083	6.561	0.425	4.43	0.00	0	4.43	2.185	0.135	2.03	2.03
14	28.086	6.557	0.441	4.37	0.00	0	4.37	2.203	0.140	1.98	1.98
14.5	29.089	6.556	0.457	4.36	0.00	0	4.36	2.222	0.145	1.96	1.96
15	30.092	6.567	0.472	4.52	0.00	0	4.52	2.240	0.150	2.02	2.02
											0.00

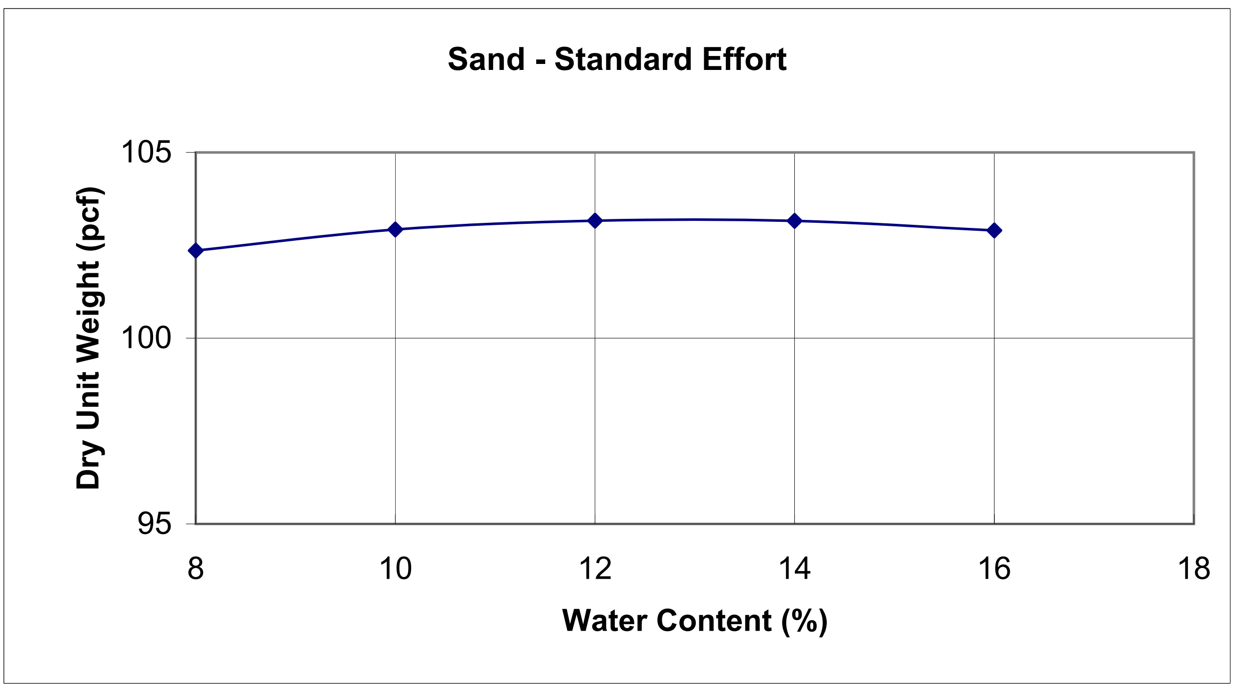
Compaction Test for the BEST Test

Test Date: 5/15/2002

Soil Type: Sand (Soil 1)

Compaction Effort: Standard

Water Content (%)	8	10	12	14	16	
Mold No.	5	5	5	5	5	
Wt of Mold (g)	6810	6810	6810	6810	6810	
Vol of Mold (cm^3)	2124	2124	2124	2124	2124	
Wt of (S.+M.) (g)	10571	10662	10741	10811	10871	
Wt of Soil (g)	3761	3852	3931	4001	4061	
Total Unit Wt (t/m^3)	1.7707	1.8136	1.8508	1.8837	1.9120	
Dry Unit Wt (t/m^3)	1.6396	1.6487	1.6525	1.6524	1.6482	
Dry Unit Wt (pcf)	102.36	102.93	103.16	103.16	102.90	



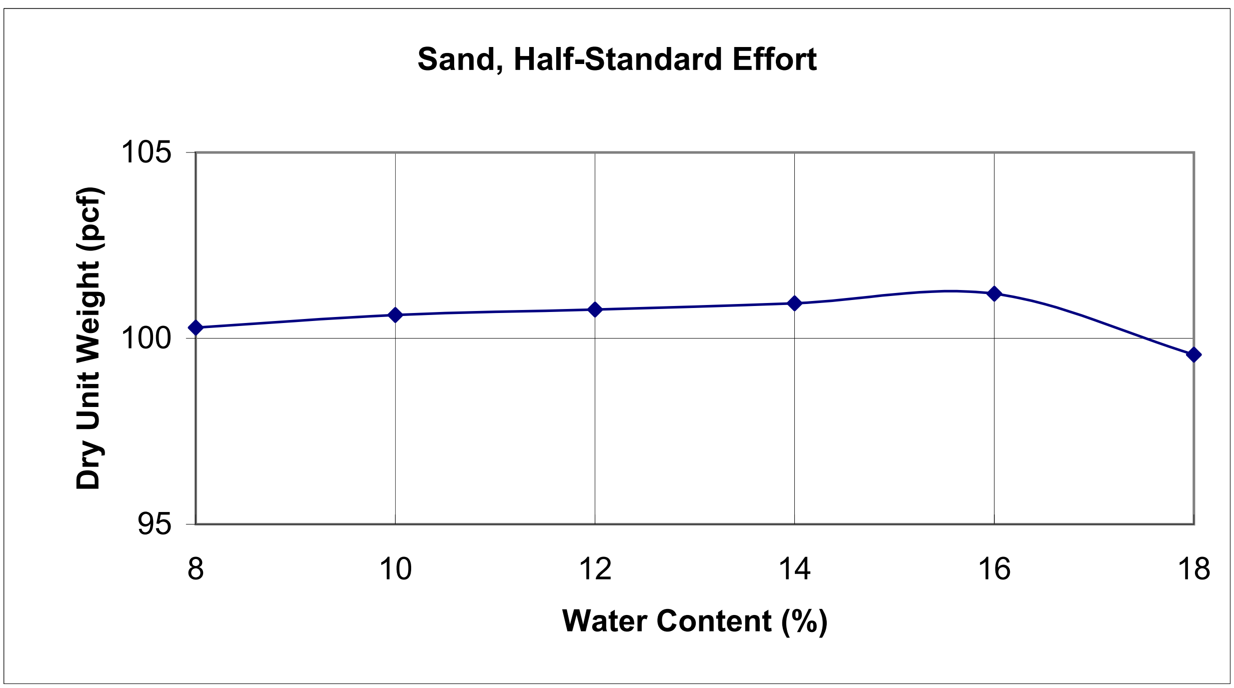
Compaction Test for the BEST Test

Test Date: 5/15/2002

Soil Type: Sand (Soil 1)

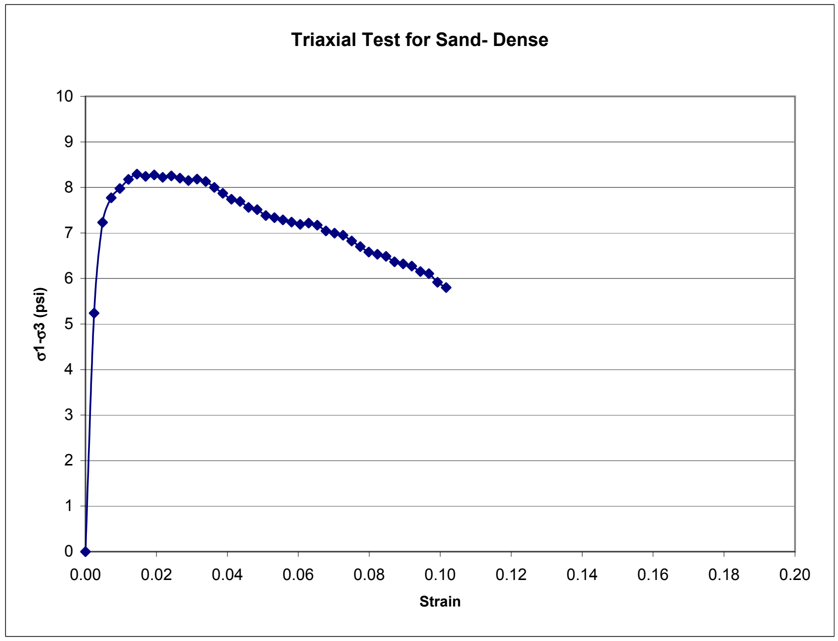
Compaction Effort: Half-Standard

Water Content (%)	8	10	12	14	16	18
Mold No.	6	6	6	6	6	6
Wt of Mold (g)	6871	6871	6871	6871	6871	6871
Vol of Mold (cm^3)	2124	2124	2124	2124	2124	2124
Wt of (S.+M.) (g)	10556	10637	10711	10786	10865	10868
Wt of Soil (g)	3685	3766	3840	3915	3994	3997
Total Unit Wt (t/m^3)	1.7349	1.7731	1.8079	1.8432	1.8804	1.8818
Dry Unit Wt (t/m^3)	1.6064	1.6119	1.6142	1.6169	1.6210	1.5948
Dry Unit Wt (pcf)	100.29	100.63	100.77	100.94	101.20	99.56



Sample No. **Sand for BEST - Dense**

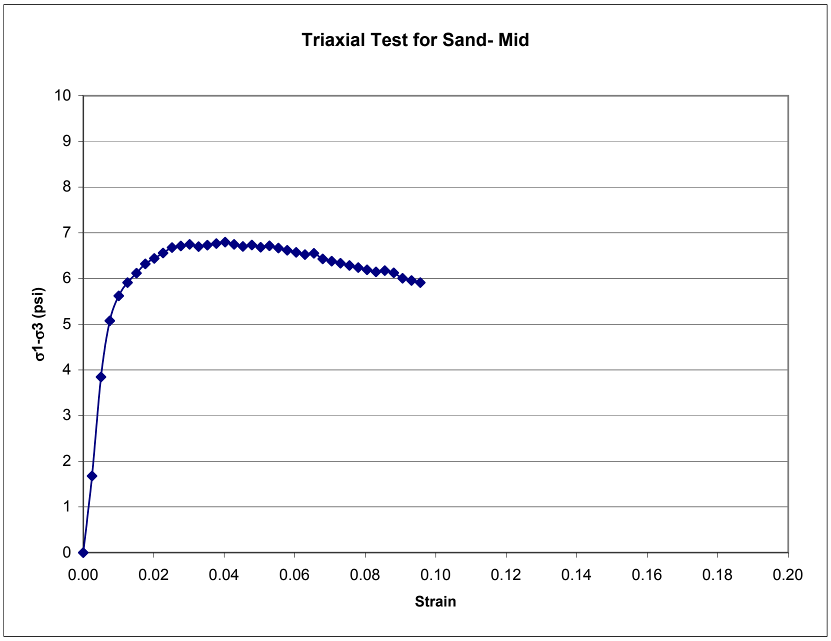
Depth		ft
Unit Weight	107.5	lb/ft^3
W/C	0	%
σ_3	5	psi
Strain Rate	0.38	mm/min
Dia. of Sample	1.5	in
L. of Sample	3.1	in
Weight	154.1	g
# Dis Trans:	9	y=0.015632x+0.014741
# Force Trans:	CZ0245	y=15.047343x-3.890211



Time (min)	Disp. (mV)	Force (mV)	Disp. (in)	Load Cell f (lb)	Sig_a*(As-Ap) (lb)	Wp (lb)	Total F (lb)	Corrected A (in^2)	Total Stress (psi)	Strain (in/in)	σ_1-σ_3 (psi)
0	0.000	0.4	0.000	2.20	6.64	1.05	9.89	1.767	5.59	0	0.00
0.5	0.479	1.02	0.007	11.53	6.64	1.05	19.21	1.774	10.83	0.00242	5.24
1	0.957	1.26	0.015	15.14	6.64	1.05	22.83	1.780	12.82	0.004841	7.23
1.5	1.436	1.33	0.022	16.19	6.64	1.05	23.88	1.786	13.37	0.007261	7.77
2	1.914	1.36	0.030	16.65	6.64	1.05	24.33	1.793	13.57	0.009682	7.98
2.5	2.393	1.39	0.037	17.10	6.64	1.05	24.78	1.800	13.77	0.012102	8.18
3	2.871	1.41	0.045	17.40	6.64	1.05	25.08	1.806	13.89	0.014522	**8.29**
3.5	3.350	1.41	0.052	17.40	6.64	1.05	25.08	1.813	13.84	0.016943	8.24
4	3.828	1.42	0.060	17.55	6.64	1.05	25.23	1.820	13.87	0.019363	8.27
4.5	4.307	1.42	0.067	17.55	6.64	1.05	25.23	1.826	13.82	0.021783	8.22
5	4.785	1.43	0.075	17.70	6.64	1.05	25.38	1.833	13.85	0.024204	8.25
5.5	5.264	1.43	0.082	17.70	6.64	1.05	25.38	1.840	13.80	0.026624	8.20
6	5.742	1.43	0.090	17.70	6.64	1.05	25.38	1.847	13.75	0.029045	8.15
6.5	6.221	1.44	0.097	17.85	6.64	1.05	25.53	1.853	13.78	0.031465	8.18
7	6.699	1.44	0.105	17.85	6.64	1.05	25.53	1.860	13.73	0.033885	8.13
7.5	7.178	1.43	0.112	17.70	6.64	1.05	25.38	1.867	13.59	0.036306	8.00
8	7.656	1.42	0.120	17.55	6.64	1.05	25.23	1.874	13.46	0.038726	7.87
8.5	8.135	1.41	0.127	17.40	6.64	1.05	25.08	1.881	13.33	0.041146	7.74
9	8.613	1.41	0.135	17.40	6.64	1.05	25.08	1.888	13.28	0.043567	7.69
9.5	9.092	1.4	0.142	17.25	6.64	1.05	24.93	1.895	13.15	0.045987	7.56
10	9.571	1.4	0.150	17.25	6.64	1.05	24.93	1.902	13.11	0.048408	7.51
10.5	10.049	1.39	0.157	17.10	6.64	1.05	24.78	1.910	12.98	0.050828	7.38
11	10.528	1.39	0.165	17.10	6.64	1.05	24.78	1.917	12.93	0.053248	7.33
11.5	11.006	1.39	0.172	17.10	6.64	1.05	24.78	1.924	12.88	0.055669	7.29
12	11.485	1.39	0.180	17.10	6.64	1.05	24.78	1.931	12.83	0.058089	7.24
12.5	11.963	1.39	0.187	17.10	6.64	1.05	24.78	1.939	12.78	0.06051	7.19
13	12.442	1.4	0.194	17.25	6.64	1.05	24.93	1.946	12.81	0.06293	7.22
13.5	12.920	1.4	0.202	17.25	6.64	1.05	24.93	1.953	12.76	0.06535	7.17
14	13.399	1.39	0.209	17.10	6.64	1.05	24.78	1.961	12.64	0.067771	7.04
14.5	13.877	1.39	0.217	17.10	6.64	1.05	24.78	1.968	12.59	0.070191	7.00
15	14.356	1.39	0.224	17.10	6.64	1.05	24.78	1.976	12.54	0.072611	6.95
15.5	14.834	1.38	0.232	16.95	6.64	1.05	24.63	1.983	12.42	0.075032	6.83
16	15.313	1.37	0.239	16.80	6.64	1.05	24.48	1.991	12.30	0.077452	6.70
16.5	15.791	1.36	0.247	16.65	6.64	1.05	24.33	1.999	12.17	0.079873	6.58
17	16.270	1.36	0.254	16.65	6.64	1.05	24.33	2.006	12.13	0.082293	6.53
17.5	16.748	1.36	0.262	16.65	6.64	1.05	24.33	2.014	12.08	0.084713	6.49
18	17.227	1.35	0.269	16.49	6.64	1.05	24.18	2.022	11.96	0.087134	6.37
18.5	17.705	1.35	0.277	16.49	6.64	1.05	24.18	2.030	11.91	0.089554	6.32
19	18.184	1.35	0.284	16.49	6.64	1.05	24.18	2.038	11.87	0.091975	6.27
19.5	18.663	1.34	0.292	16.34	6.64	1.05	24.03	2.045	11.75	0.094395	6.15
20	19.141	1.34	0.299	16.34	6.64	1.05	24.03	2.053	11.70	0.096815	6.11
20.5	19.620	1.32	0.307	16.04	6.64	1.05	23.73	2.061	11.51	0.099236	5.92
21	20.098	1.31	0.314	15.89	6.64	1.05	23.58	2.069	11.39	0.101656	5.80

Sample No. **Sand for BEST - Mid**

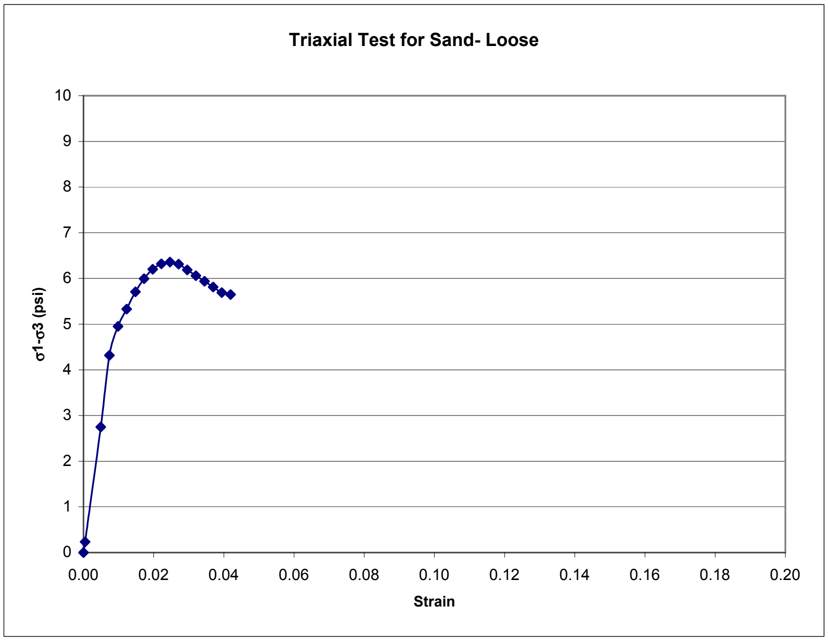
Depth		ft
Unit Weight	105.5	lb/ft^3
W/C	0	%
σ_3	5	psi
Strain Rate	0.38	mm/min
Dia. of Sample	1.5	in
L. of Sample	3.0	in
Weight	145.5	g
# Dis Trans:	9	y=0.015632x+0.014741
# Force Trans:	CZ0245	y=15.047343x-3.890211



Time (min)	Disp. (mV)	Force (mV)	Disp. (in)	Load Cell f (lb)	Sig_a*(As-Ap) (lb)	Wp (lb)	Total F (lb)	Corrected A (in^2)	Total Stress (psi)	Strain (in/in)	σ_1-σ_3 (psi)
0	0.000	0.4	0.000	2.20	6.64	1.05	9.89	1.767	5.59	0	0.00
0.5	0.479	0.6	0.007	5.21	6.64	1.05	12.89	1.774	7.27	0.002517	1.68
1	0.957	0.86	0.015	9.12	6.64	1.05	16.81	1.781	9.44	0.005033	3.85
1.5	1.436	1.01	0.022	11.38	6.64	1.05	19.06	1.787	10.67	0.00755	5.07
2	1.914	1.08	0.030	12.43	6.64	1.05	20.12	1.794	11.21	0.010066	5.62
2.5	2.393	1.12	0.037	13.03	6.64	1.05	20.72	1.801	11.51	0.012583	5.91
3	2.871	1.15	0.045	13.49	6.64	1.05	21.17	1.808	11.71	0.015099	6.12
3.5	3.350	1.18	0.052	13.94	6.64	1.05	21.62	1.815	11.92	0.017616	6.32
4	3.828	1.2	0.060	14.24	6.64	1.05	21.92	1.822	12.03	0.020132	6.44
4.5	4.307	1.22	0.067	14.54	6.64	1.05	22.22	1.829	12.15	0.022649	6.56
5	4.785	1.24	0.075	14.84	6.64	1.05	22.53	1.836	12.27	0.025166	6.68
5.5	5.264	1.25	0.082	14.99	6.64	1.05	22.68	1.843	12.31	0.027682	6.71
6	5.742	1.26	0.090	15.14	6.64	1.05	22.83	1.850	12.34	0.030199	6.75
6.5	6.221	1.26	0.097	15.14	6.64	1.05	22.83	1.857	12.29	0.032715	6.70
7	6.699	1.27	0.105	15.29	6.64	1.05	22.98	1.864	12.33	0.035232	6.73
7.5	7.178	1.28	0.112	15.44	6.64	1.05	23.13	1.871	12.36	0.037748	6.76
8	7.656	1.29	0.120	15.59	6.64	1.05	23.28	1.879	12.39	0.040265	**6.80**
8.5	8.135	1.29	0.127	15.59	6.64	1.05	23.28	1.886	12.34	0.042781	6.75
9	8.613	1.29	0.135	15.59	6.64	1.05	23.28	1.893	12.29	0.045298	6.70
9.5	9.092	1.3	0.142	15.74	6.64	1.05	23.43	1.901	12.33	0.047815	6.73
10	9.571	1.3	0.150	15.74	6.64	1.05	23.43	1.908	12.28	0.050331	6.68
10.5	10.049	1.31	0.157	15.89	6.64	1.05	23.58	1.916	12.31	0.052848	6.71
11	10.528	1.31	0.165	15.89	6.64	1.05	23.58	1.923	12.26	0.055364	6.67
11.5	11.006	1.31	0.172	15.89	6.64	1.05	23.58	1.931	12.21	0.057881	6.62
12	11.485	1.31	0.180	15.89	6.64	1.05	23.58	1.938	12.16	0.060397	6.57
12.5	11.963	1.31	0.187	15.89	6.64	1.05	23.58	1.946	12.12	0.062914	6.52
13	12.442	1.32	0.194	16.04	6.64	1.05	23.73	1.954	12.15	0.06543	6.55
13.5	12.920	1.31	0.202	15.89	6.64	1.05	23.58	1.961	12.02	0.067947	6.43
14	13.399	1.31	0.209	15.89	6.64	1.05	23.58	1.969	11.97	0.070464	6.38
14.5	13.877	1.31	0.217	15.89	6.64	1.05	23.58	1.977	11.93	0.07298	6.33
15	14.356	1.31	0.224	15.89	6.64	1.05	23.58	1.985	11.88	0.075497	6.29
15.5	14.834	1.31	0.232	15.89	6.64	1.05	23.58	1.993	11.83	0.078013	6.24
16	15.313	1.31	0.239	15.89	6.64	1.05	23.58	2.001	11.79	0.08053	6.19
16.5	15.791	1.31	0.247	15.89	6.64	1.05	23.58	2.009	11.74	0.083046	6.14
17	16.270	1.32	0.254	16.04	6.64	1.05	23.73	2.017	11.77	0.085563	6.17
17.5	16.748	1.32	0.262	16.04	6.64	1.05	23.73	2.025	11.72	0.088079	6.12
18	17.227	1.31	0.269	15.89	6.64	1.05	23.58	2.033	11.60	0.090596	6.00
18.5	17.705	1.31	0.277	15.89	6.64	1.05	23.58	2.041	11.55	0.093113	5.96
19	18.184	1.31	0.284	15.89	6.64	1.05	23.58	2.049	11.50	0.095629	5.91

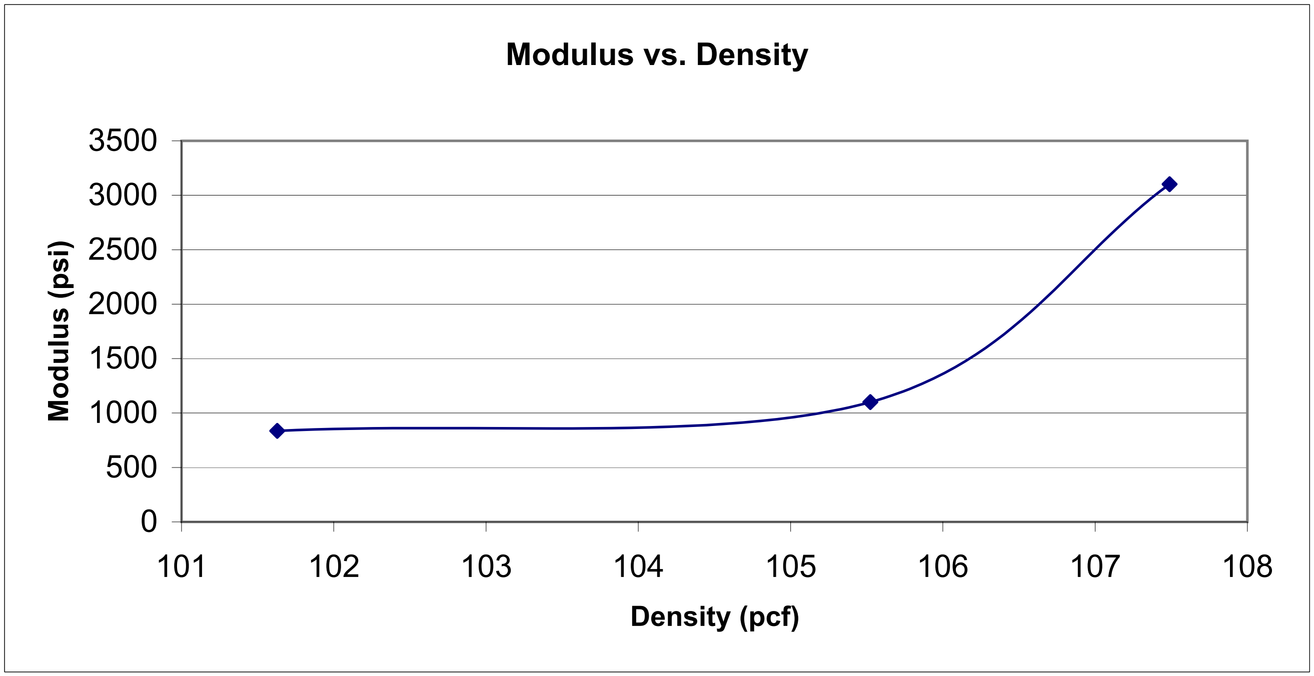
Sample No. **Sand for BEST - Loose**

Depth		ft
Unit Weight	101.6	lb/ft^3
W/C	0	%
σ_3	5	psi
Strain Rate	0.38	mm/min
Dia. of Sample	1.5	in
L. of Sample	3.0	in
Weight	143.1	g
# Dis Trans:	9	y=0.015632x+0.014741
# Force Trans:	CZ0245	y=15.047343x-3.890211



Time (min)	Disp. (mV)	Force (mV)	Disp. (in)	Load Cell f (lb)	Sig_a*(As-Ap) (lb)	Wp (lb)	Total F (lb)	Corrected A (in^2)	Total Stress (psi)	Strain (in/in)	σ_1-σ_3 (psi)
0	0.000	0.4	0.000	2.20	6.64	1.05	9.89	1.767	5.59	0	0.00
0.5	0.479	0.43	0.007	2.65	6.64	1.05	10.34	1.774	5.83	0.000464	0.23
1	0.957	0.73	0.015	7.17	6.64	1.05	14.85	1.780	8.34	0.004929	2.75
1.5	1.436	0.92	0.022	10.02	6.64	1.05	17.71	1.787	9.91	0.007393	4.32
2	1.914	1	0.030	11.23	6.64	1.05	18.91	1.794	10.55	0.009857	4.95
2.5	2.393	1.05	0.037	11.98	6.64	1.05	19.67	1.800	10.92	0.012322	5.33
3	2.871	1.1	0.045	12.73	6.64	1.05	20.42	1.807	11.30	0.014786	5.71
3.5	3.350	1.14	0.052	13.34	6.64	1.05	21.02	1.814	11.59	0.01725	6.00
4	3.828	1.17	0.060	13.79	6.64	1.05	21.47	1.820	11.79	0.019715	6.20
4.5	4.307	1.19	0.067	14.09	6.64	1.05	21.77	1.827	11.92	0.022179	6.32
5	4.785	1.2	0.075	14.24	6.64	1.05	21.92	1.834	11.95	0.024643	**6.36**
5.5	5.264	1.2	0.082	14.24	6.64	1.05	21.92	1.841	11.91	0.027108	6.31
6	5.742	1.19	0.090	14.09	6.64	1.05	21.77	1.848	11.78	0.029572	6.19
6.5	6.221	1.18	0.097	13.94	6.64	1.05	21.62	1.855	11.66	0.032036	6.06
7	6.699	1.17	0.105	13.79	6.64	1.05	21.47	1.862	11.53	0.034501	5.94
7.5	7.178	1.16	0.112	13.64	6.64	1.05	21.32	1.869	11.41	0.036965	5.81
8	7.656	1.15	0.120	13.49	6.64	1.05	21.17	1.876	11.28	0.039429	5.69
8.5	8.135	1.15	0.127	13.49	6.64	1.05	21.17	1.883	11.24	0.041894	5.65

	Density (pcf)	$(\sigma1-\sigma3)_{max}$ (psi)	$(\sigma1-\sigma3)_{max}/2$ (psi)	ε_{50}	σ_1 (psi)	E_{50} (psi)
Dense	107.5	8.29	4.147	0.00182	9.147	3102.5
Mid	105.5	6.80	3.398	0.00445	8.398	1100.7
Loose	101.6	6.36	3.179	0.00560	8.179	835.6



$E_{50} = (\sigma1-0.7\sigma3)/\varepsilon_{50}$

APPENDIX I BEST DEVICE TEST RESULTS

Table I-1 Test 1.

Length (ft)	Cycles (No.)						
	Initial	100	1000	10000	50000	100000	200000
0	-6.29	-6.26	-6.22	-6.08	-5.92	-5.83	-5.60
1.59	-6.20	-6.17	-6.11	-5.97	-5.62	-5.60	-5.44
3.19	-5.87	-5.82	-5.72	-5.67	-5.65	-5.61	-5.51
3.69	-6.99	-6.97	-6.95	-6.93	-6.89	-6.88	-6.70
4.22	-6.10	-6.05	-6.07	-6.07	-6.09	-6.08	-6.07
6.31	-9.60	-9.59	-9.62	-9.61	-9.62	-9.64	-9.61
8.38	-6.18	-6.19	-6.18	-6.18	-6.19	-6.19	-6.19
8.88	-7.61	-7.59	-7.59	-7.57	-7.50	-7.40	-7.25
9.38	-6.38	-6.36	-6.35	-6.28	-6.14	-6.06	-5.81
10.97	-5.93	-5.89	-5.84	-5.74	-5.61	-5.59	-5.40

Table I-2. Test 2.

Length (ft)	Cycles (No.)					
	Initial	1000	10000	50000	100000	200000
0	-5.51	-5.31	-5.24	-5.47	-5.53	-5.72
1.59	-5.50	-5.55	-5.41	-5.42	-5.38	-5.59
3.19	-5.63	-5.48	-5.36	-5.29	-5.23	-5.19
3.69	-7.64	-7.38	-7.33	-7.26	-7.24	-7.21
4.22	-6.63	-6.49	-6.62	-6.63	-6.63	-6.55
5.27	-4.69	-4.68	-4.61	-4.63	-4.48	-4.39
6.31	-3.13	-3.07	-3.19	-3.10	-3.09	-2.94
7.36	-5.01	-4.92	-4.98	-4.96	-4.91	-4.79
8.38	-6.56	-6.58	-6.60	-6.58	-6.59	-6.61
8.88	-6.63	-6.59	-6.44	-6.10	-5.98	-5.94
9.38	-4.70	-4.55	-4.26	-3.46	-3.08	-2.85
10.97	-5.46	-5.15	-4.85	-4.66	-4.60	-4.45

Table I-3. Test 3.

Length (ft)	Cycles (No.)						
	Initial	100	1000	10000	50000	150000	200000
0	-5.94	-5.91	-5.87	-5.65	-5.60	-5.45	-5.38
1.59	-5.50	-5.38	-5.30	-5.26	-5.25	-5.15	-5.08
3.19	-5.10	-5.08	-5.05	-5.03	-5.01	-4.90	-4.82
3.69	-7.58	-7.56	-7.52	-7.50	-7.46	-7.33	-7.28
4.22	-6.70	-6.70	-6.70	-6.70	-6.70	-6.70	-6.69
6.31	-7.38	-7.36	-7.37	-7.37	-7.37	-7.37	-7.37
8.38	-7.03	-7.02	-7.03	-7.03	-7.03	-7.03	-7.02
8.88	-8.00	-7.86	-7.63	-7.57	-7.43	-7.21	-7.07
9.38	-4.66	-4.61	-4.48	-4.34	-4.20	-4.05	-3.91
10.97	-5.32	-5.12	-4.88	-4.60	-4.45	-4.13	-3.98

Table I-4. Test 4.

Length (ft)	Cycles (No.)						
	Initial	100	1000	10000	50000	100000	200000
0	-5.22	-5.07	-5.00	-4.88	-4.87	-4.73	-4.50
1.59	-5.09	-4.84	-4.80	-4.68	-4.51	-4.47	-4.37
3.19	-4.07	-4.00	-3.97	-3.86	-3.81	-3.73	-3.64
3.69	-5.91	-5.83	-5.78	-5.74	-5.67	-5.66	-5.62
4.22	-5.70	-5.70	-5.69	-5.68	-5.69	-5.69	-5.64
6.31	-8.79	-8.74	-8.75	-8.74	-8.74	-8.75	-8.74
8.38	-5.74	-5.74	-5.75	-5.75	-5.75	-5.75	-5.76
8.88	-6.31	-6.20	-6.05	-5.89	-5.24	-5.08	-4.99
9.38	-4.27	-4.10	-4.07	-3.83	-2.90	-2.54	-2.38
10.97	-4.72	-4.64	-4.61	-4.58	-4.49	-4.44	-4.41

Table I-5. Test 5.

Length (ft)	Cycles (No.)									
	Initial	50	250	500	2500	5000	25000	50000	250000	500000
0	-5.89	-5.73	-5.68	-5.67	-5.50	-5.46	-5.39	-5.37	-5.26	-5.02
1.59	-5.67	-5.70	-5.59	-5.54	-5.52	-5.51	-5.45	-5.43	-5.35	-5.26
3.19	-4.00	-3.88	-3.76	-3.71	-3.48	-3.26	-3.08	-3.01	-2.90	-2.86
3.69	-6.46	-6.33	-6.30	-6.23	-5.96	-5.96	-5.85	-5.84	-5.82	-5.80
4.22	-6.31	-6.33	-6.32	-6.32	-6.35	-6.36	-6.32	-6.32	-6.32	-6.30
6.31	-8.82	-8.83	-8.82	-8.82	-8.82	-8.82	-8.82	-8.82	-8.83	-8.83
8.38	-6.47	-6.47	-6.47	-6.47	-6.47	-6.48	-6.48	-6.48	-6.48	-6.49
8.88	-6.42	-6.39	-6.33	-6.29	-6.34	-6.34	-6.18	-6.10	-6.05	-6.02
9.38	-3.84	-3.74	-3.64	-3.57	-3.48	-3.42	-3.15	-3.08	-3.06	-3.02
10.97	-5.56	-5.38	-5.21	-5.17	-4.90	-4.87	-4.85	-4.83	-4.66	-4.36

Table I-6. Test 6.

Length (ft)	Cycles (No.)						
	Initial	100	1000	10000	50000	100000	200000
0	-5.57	-5.56	-5.54	-5.52	-5.51	-5.56	-5.50
1.59	-5.97	-5.93	-5.77	-5.68	-5.66	-5.63	-5.60
3.19	-4.49	-4.27	-4.13	-4.05	-4.01	-3.99	-3.96
3.69	-6.82	-6.65	-6.47	-6.38	-6.32	-6.31	-6.30
4.22	-5.88	-5.88	-5.84	-5.84	-5.89	-5.88	-5.89
6.31	-8.86	-8.86	-8.86	-8.84	-8.85	-8.85	-8.86
8.38	-6.14	-6.14	-6.14	-6.15	-6.14	-6.14	-6.14
8.88	-6.43	-6.43	-6.39	-6.35	-6.32	-6.23	-6.07
9.38	-3.80	-3.79	-3.72	-3.70	-3.63	-3.50	-3.38
10.97	-5.14	-5.13	-5.07	-5.06	-5.05	-5.06	-5.04

Table I-7. Test 7.

Length (ft)	Cycles (No.)							
	Initial	1000	10000	50000	100000	200000	300000	400000
0	-5.76	-5.49	-5.29	-5.01	-4.79	-4.74	-4.57	-4.41
1.09	-6.26	-6.03	-5.76	-5.52	-5.32	-5.28	-5.08	-4.93
2.16	-5.56	-5.4	-5.13	-5.02	-4.74	-4.6	-4.39	-4.23
2.22	-5.79	-5.78	-5.62	-5.58	-5.48	-5.43	-5.21	-4.87
2.69	-6.77	-6.5	-6.37	-6.25	-6.18	-6.14	-5.84	-5.67
3.16	-6.36	-6.22	-6.14	-5.92	-5.81	-5.71	-5.42	-5.43
3.22	-5.53	-5.35	-5.21	-4.89	-4.62	-4.55	-4.24	-4.15
3.69	-5.75	-5.64	-5.48	-5.34	-5.23	-5.26	-5.01	-5.00
4.16	-5.82	-5.92	-5.77	-5.9	-5.98	-6.1	-5.94	-6.04
4.22	-5.78	-5.92	-5.95	-5.85	-5.98	-6.06	-5.86	-5.94
5.24	-5.16	-5.05	-5.03	-4.96	-4.9	-4.95	-4.75	-4.66

Table I-7. Test 7 (Continued).

Length (ft)	Cycles (No.)							
	Initial	1000	10000	50000	100000	200000	300000	400000
6.29	-3.94	-3.95	-3.94	-3.87	-3.88	-3.9	-3.68	-3.65
7.33	-5.62	-5.58	-5.5	-5.45	-5.38	-5.32	-5.06	-5.06
8.35	-5.92	-5.88	-5.92	-5.85	-5.9	-5.93	-5.85	-5.89
8.41	-6.43	-5.98	-5.89	-5.9	-6.04	-5.97	-5.92	-5.74
8.88	-6.15	-5.81	-5.39	-5.41	-5.25	-5.29	-5.10	-4.71
9.35	-6.01	-5.74	-5.31	-5.28	-5.25	-5.08	-4.76	-3.83
9.41	-6.5	-6.31	-6.29	-5.94	-5.87	-5.77	-5.53	-4.90
9.88	-7.09	-6.48	-6.41	-6.21	-6.18	-6.13	-6.06	-5.70
10.35	-6.47	-5.84	-5.74	-5.81	-5.83	-6.03	-5.85	-5.95
10.41	-6.45	-5.78	-5.52	-5.45	-5.3	-5.25	-4.98	-5.07
11.48	-5.93	-5.44	-5.24	-4.96	-4.76	-4.73	-4.56	-4.50

Table I-8. Test 8.

Length (ft)	Cycles (No.)					
	Initial	1000	10000	50000	100000	200000
0	-6.33	-5.87	-5.74	-5.67	-5.57	-5.48
1.09	-6.22	-5.82	-5.77	-5.69	-5.45	-5.34
2.19	-5.04	-4.63	-4.23	-3.8	-3.57	-3.32
2.69	-6.17	-5.65	-5.09	-4.17	-3.68	-3.49
3.22	-4.67	-3.51	-2.78	-1.12	-0.55	-0.38
3.69	-5.86	-5.24	-4.81	-3.86	-3.62	-3.42
4.22	-6.58	-6.6	-6.7	-6.61	-6.61	-6.56
5.24	-4.87	-4.83	-4.75	-4.64	-4.64	-4.62
6.29	-3.37	-3.28	-3.21	-3.11	-3.02	-3.06
7.33	-5.04	-5.08	-5.05	-5.01	-4.93	-4.92
8.35	-6.59	-6.59	-6.6	-6.62	-6.56	-6.52
8.88	-6.06	-5.76	-5.32	-5	-4.87	-4.66
9.35	-5.62	-5.03	-4.43	-4.01	-3.73	-3.41
9.88	-6.5	-6.23	-5.62	-5.26	-4.96	-4.55
10.38	-5.38	-5.05	-4.44	-4.08	-3.66	-3.17
11.48	-7.07	-5.89	-5.65	-5.45	-5.27	-5.21

Table I-9. Test 9.

Length (ft)	Cycles (No.)						
	Initial	100	1000	10000	50000	100000	200000
0	-4.85	-4.8	-4.68	-4.43	-4.41	-4.33	-4.23
1.09	-5.8	-5.68	-5.58	-5.47	-5.3	-5.22	-5.18
2.19	-4.95	-4.9	-4.68	-4.55	-4.41	-4.3	-4.27
2.69	-6.6	-6.41	-6.24	-6.11	-6	-5.91	-5.9
3.22	-3.56	-3.52	-3.42	-3.31	-3.28	-3.26	-3.24
3.69	-4.95	-4.94	-4.85	-4.84	-4.84	-4.84	-4.83
4.22	-6.84	-6.84	-6.84	-6.85	-6.86	-6.85	-6.85
6.29	-6.61	-6.61	-6.61	-6.62	-6.62	-6.62	-6.62
8.35	-7.12	-7.12	-7.12	-7.13	-7.12	-7.12	-7.12
8.88	-5.89	-5.83	-5.77	-5.62	-5.58	-5.54	-5.52
9.35	-4.71	-4.58	-4.41	-4.26	-4.22	-4.21	-4.18
9.88	-6.51	-6.36	-6.31	-6.17	-6.04	-6.01	-5.97
10.38	-4.29	-4.25	-4.2	-4.16	-4.15	-4.1	-4.08
11.48	-4.71	-4.66	-4.45	-4.26	-4.16	-4.07	-4.06

Table I-10. Test 10.

Length (ft)	Cycles (No.)							
	Initial	50	250	500	2500	5000	9000	100000
0	-5.18	-5.04	-4.93	-4.84	-4.71	-4.59	-4.57	-3.93
1.09	-5.38	-5.28	-5.24	-5.17	-5.12	-5.09	-5.07	-4.64
2.19	-4.11	-4.06	-4.02	-4.00	-3.87	-3.85	-3.83	-3.72
3.22	-3.76	-3.73	-3.70	-3.60	-3.38	-3.32	-3.30	-3.06
4.22	-7.23	-7.23	-7.23	-7.23	-7.23	-7.23	-7.23	-7.23
6.29	-4.15	-4.15	-4.15	-4.15	-4.15	-4.15	-4.15	-4.15
8.35	-7.46	-7.46	-7.46	-7.46	-7.46	-7.46	-7.46	-7.46
9.35	-5.45	-5.41	-5.29	-5.24	-4.97	-4.83	-4.82	-3.66
10.38	-5.50	-5.47	-5.40	-5.38	-5.13	-5.06	-5.03	-4.62
11.48	-5.26	-5.02	-4.88	-4.81	-4.55	-4.43	-4.41	-3.84

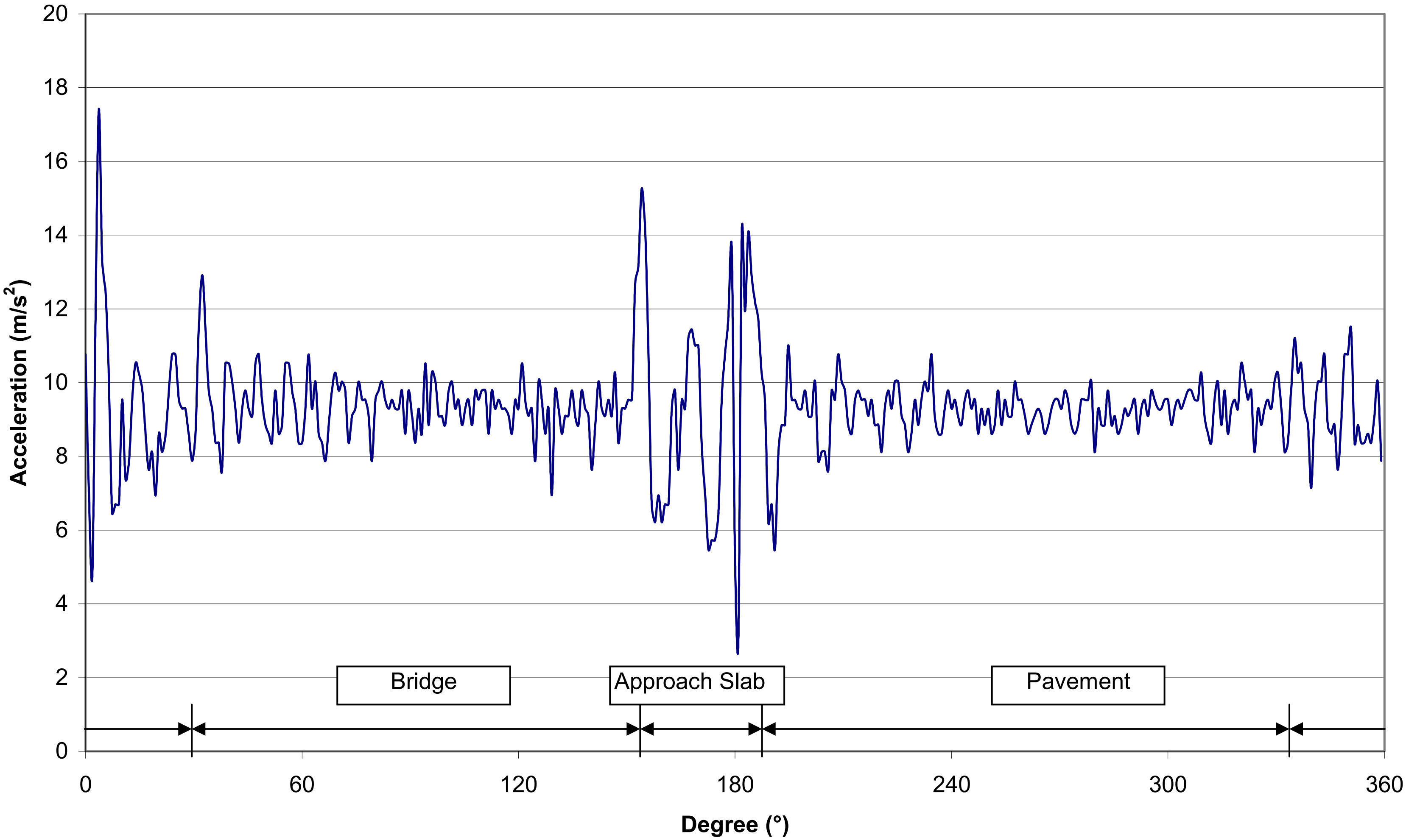


Figure I-1. Test 3 – Acceleration (Cycle No. 1).

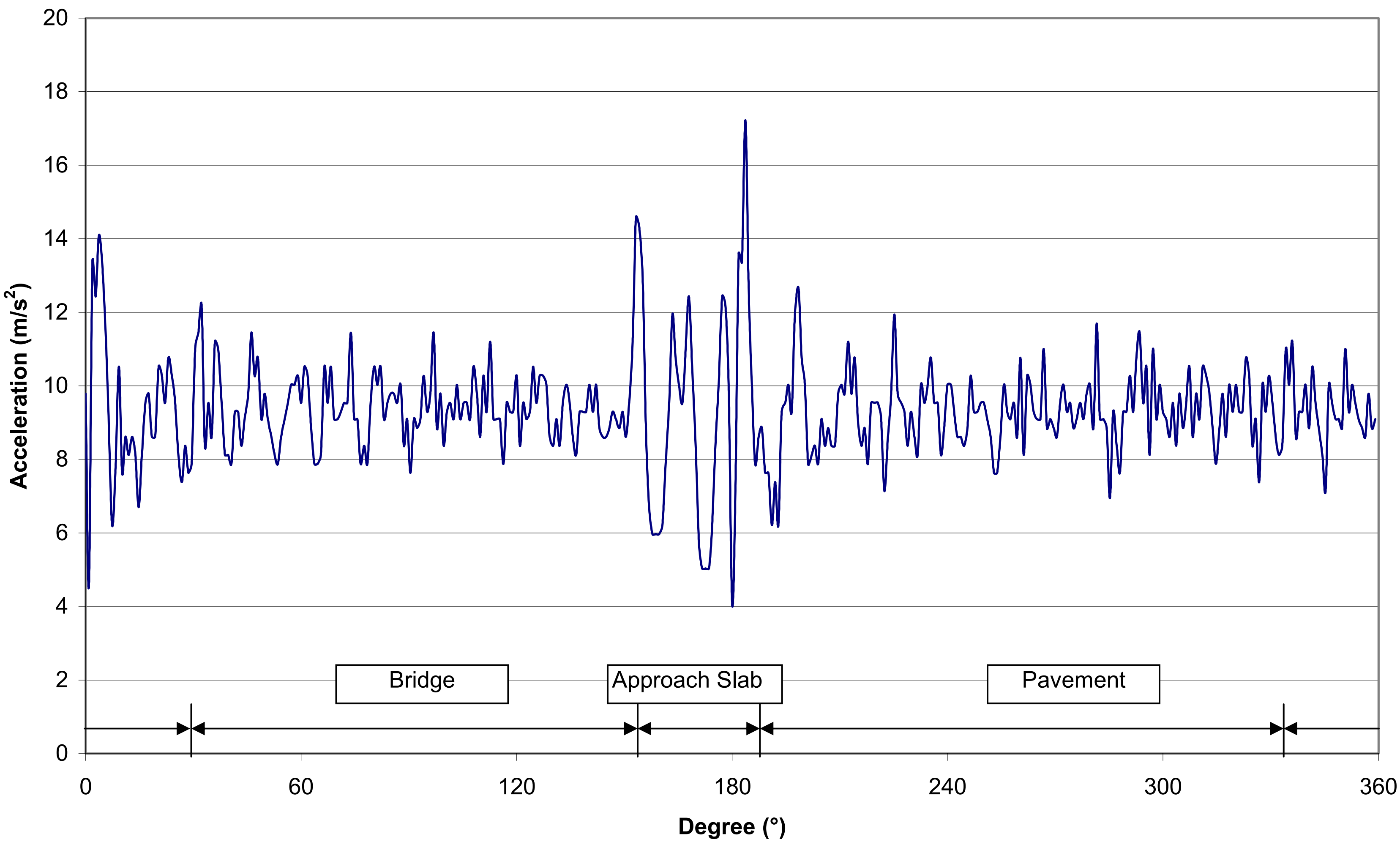


Figure I-2. Test 3 – Acceleration (Cycle No. 100).

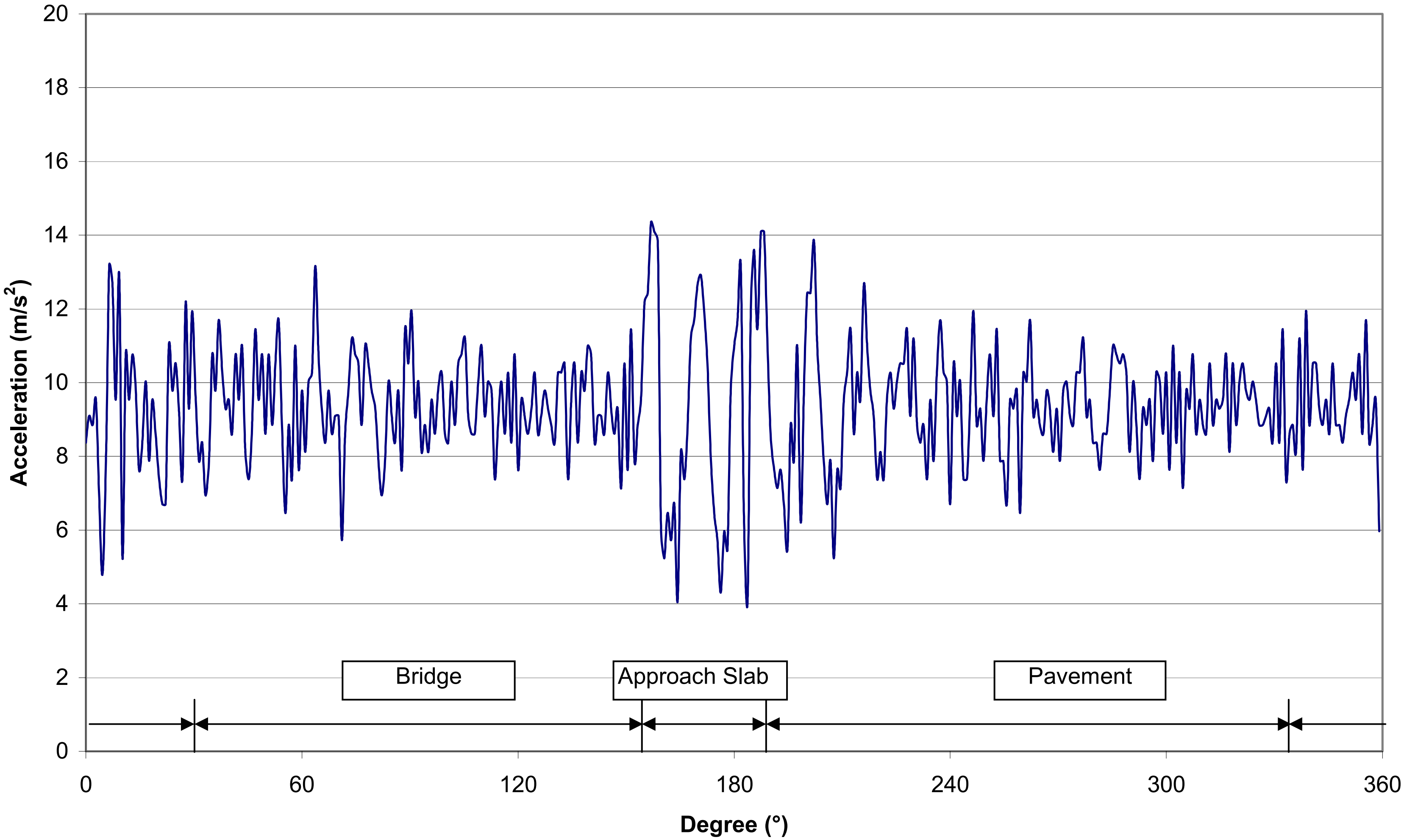


Figure I-3. Test 3 – Acceleration (Cycle No. 1000).

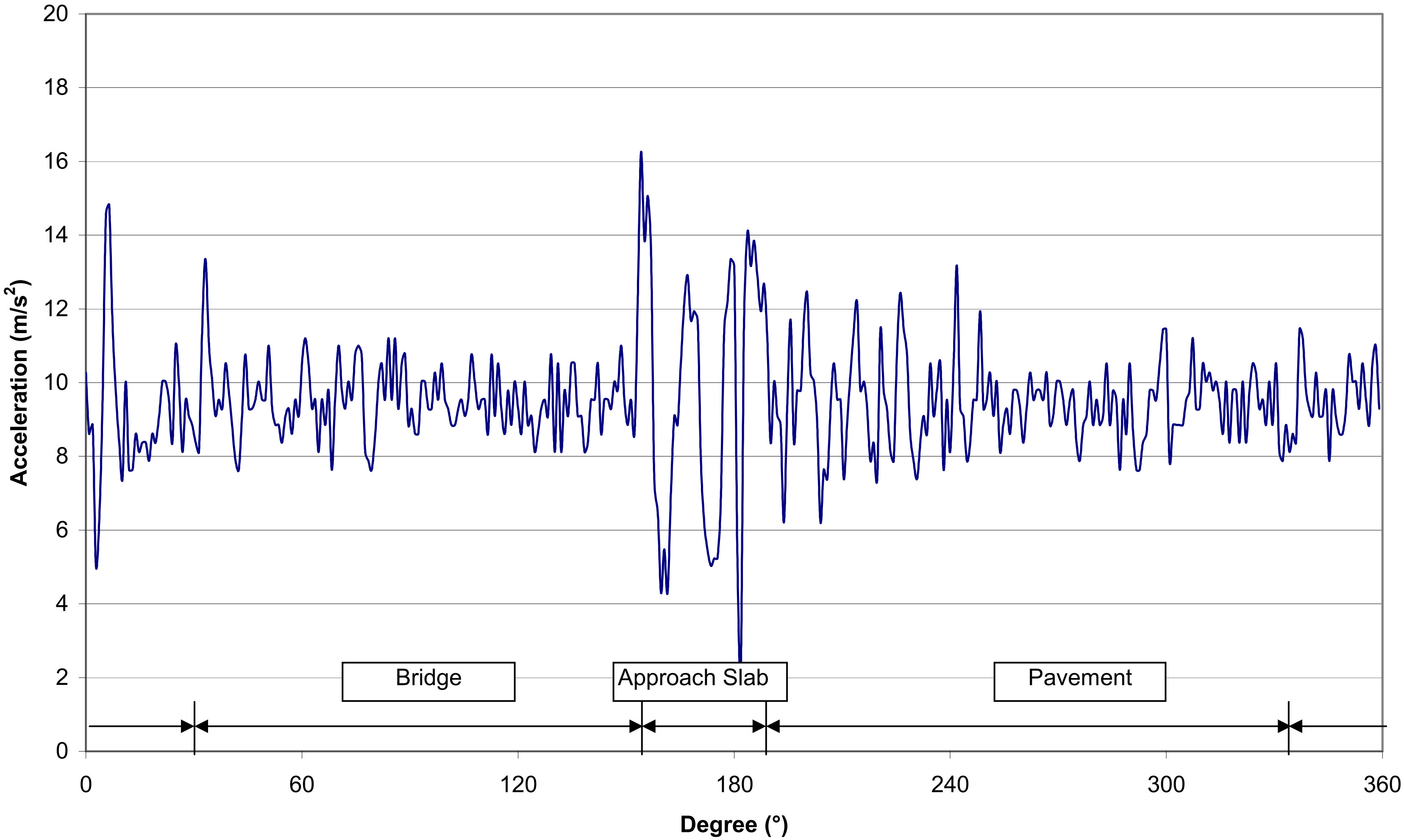


Figure I-4. Test 3 – Acceleration (Cycle No. 10,000).

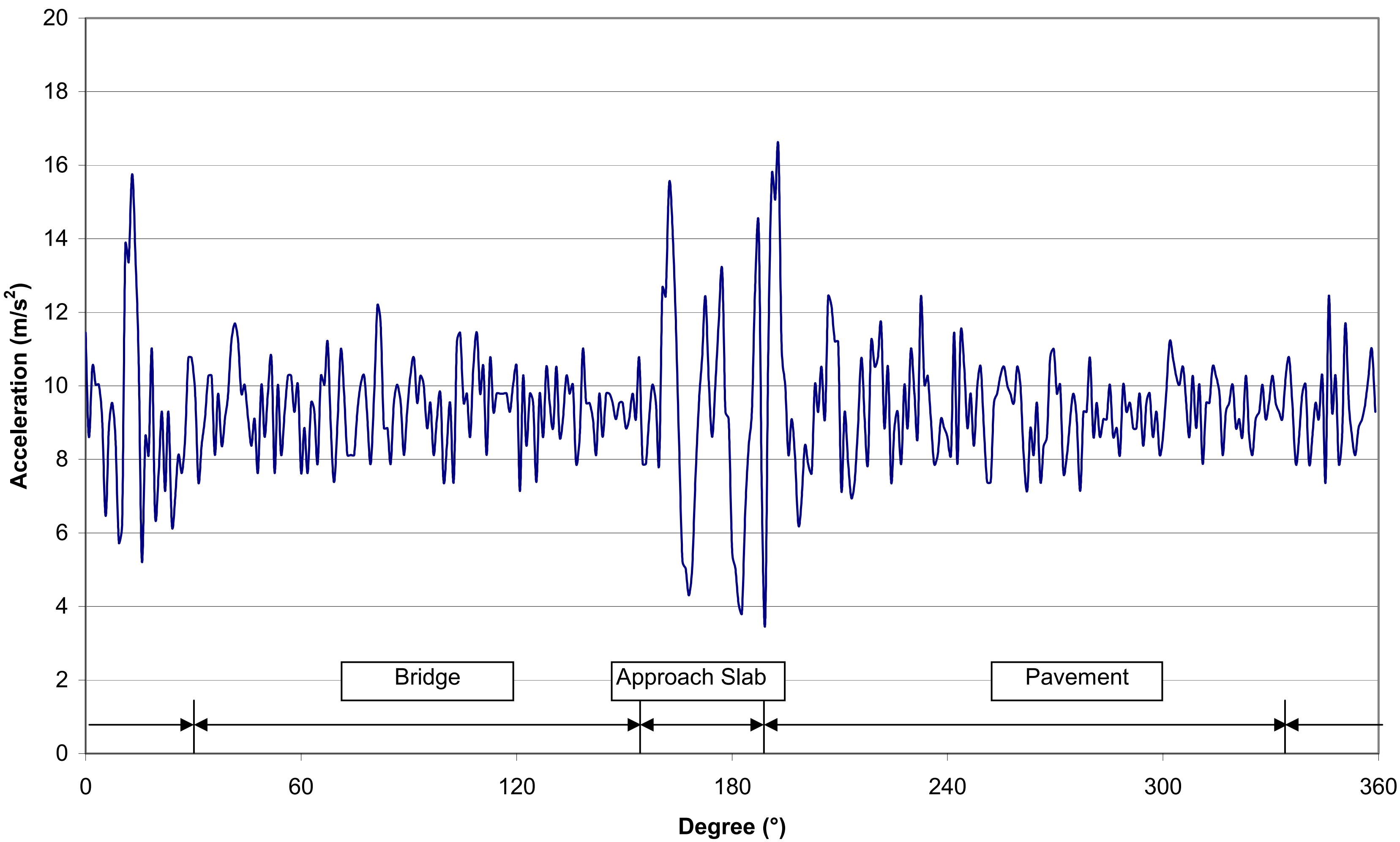


Figure I-5. Test 3 – Acceleration (Cycle No. 50,000).

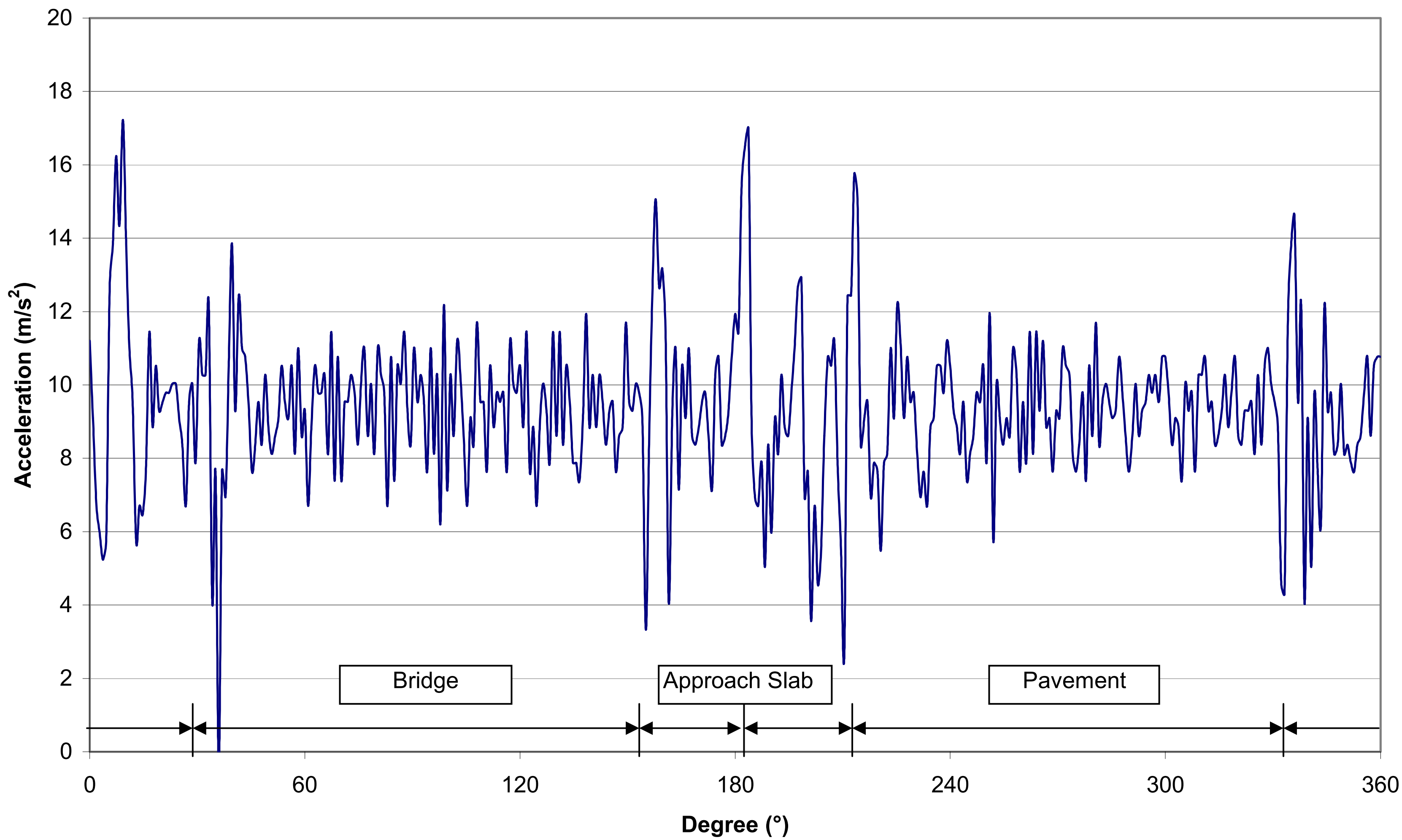


Figure I-6. Test 9 – Acceleration (Cycle No. 100).

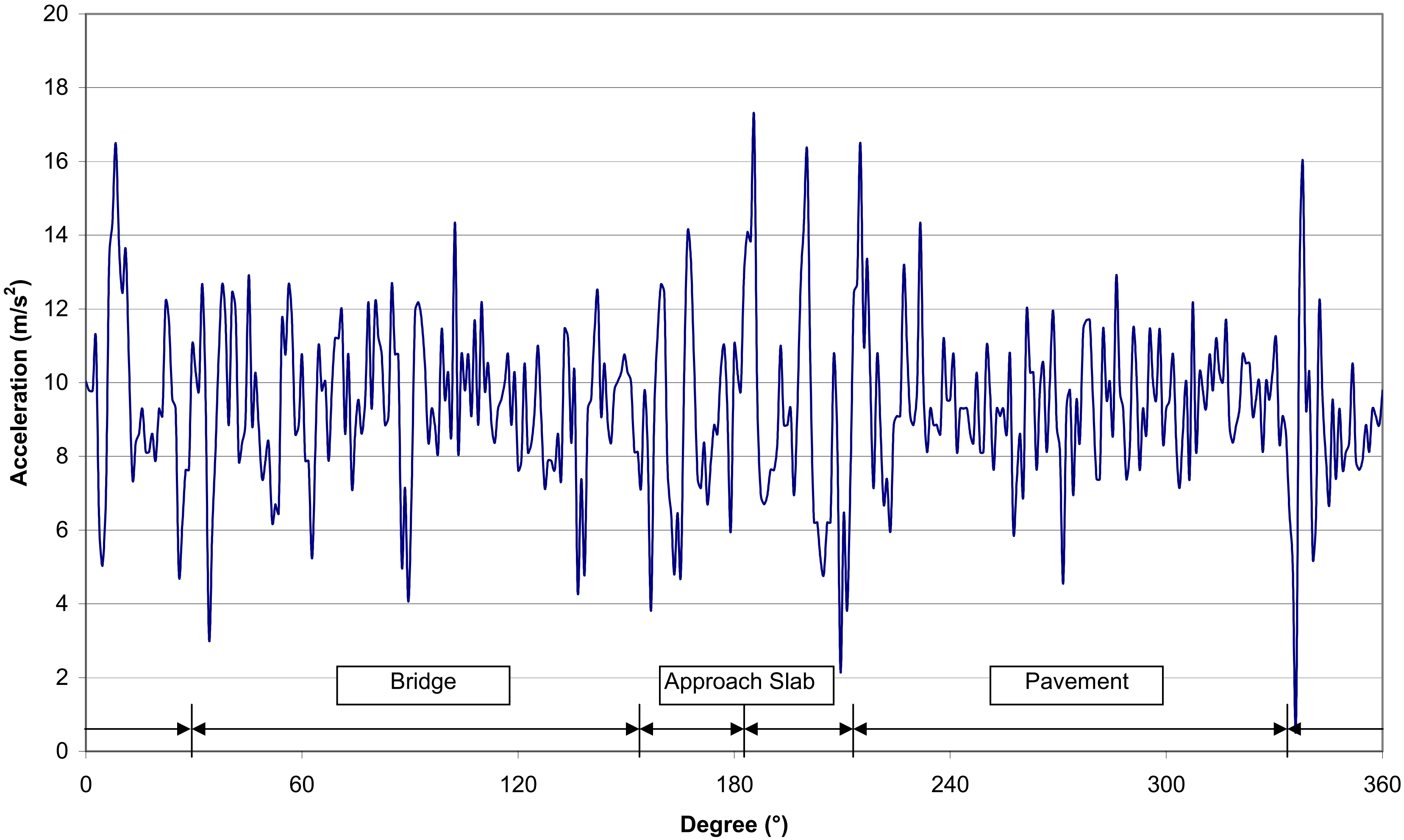


Figure I-7. Test 9 – Acceleration (Cycle No. 1,000).

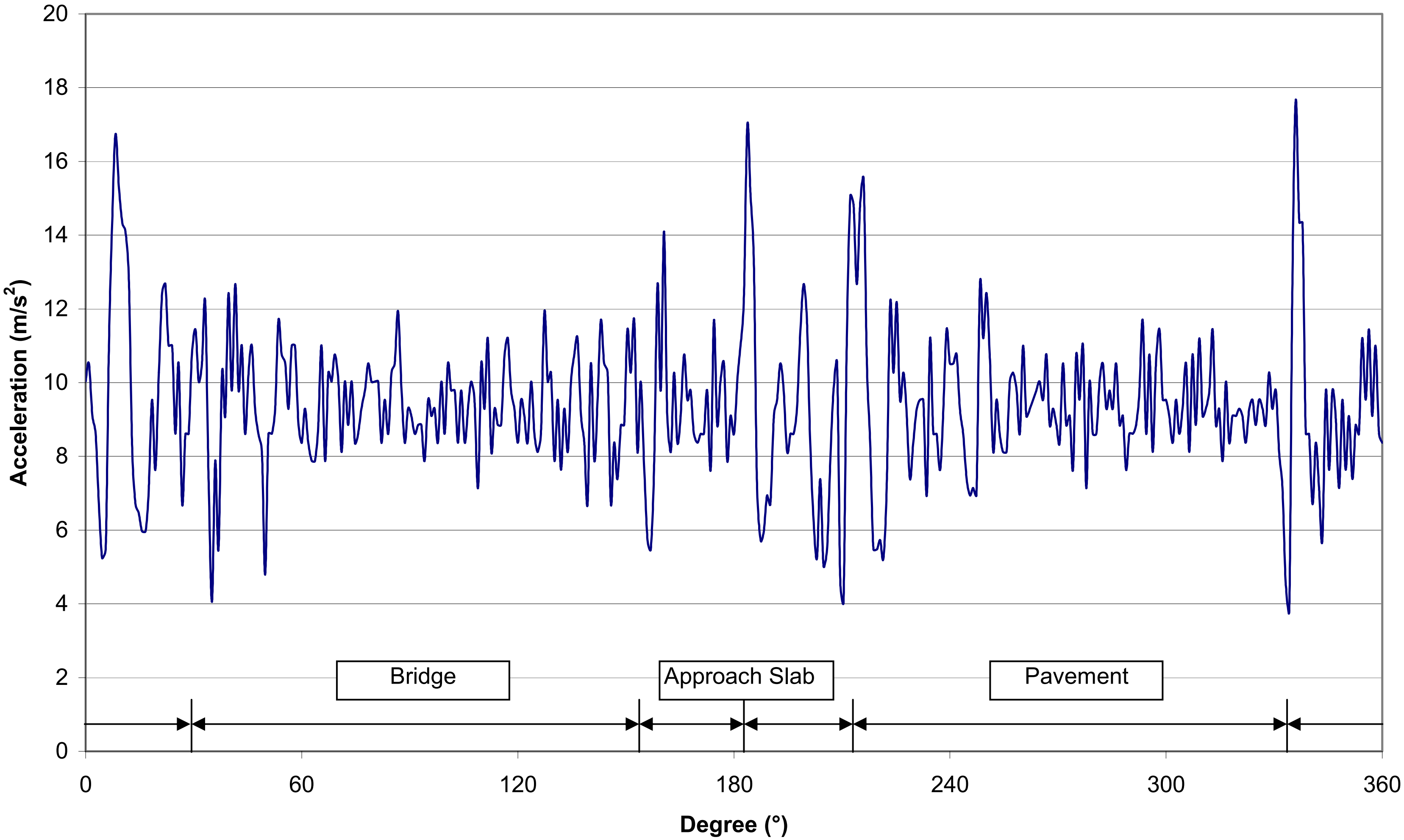


Figure I-8. Test 9 – Acceleration (Cycle No. 10,000).

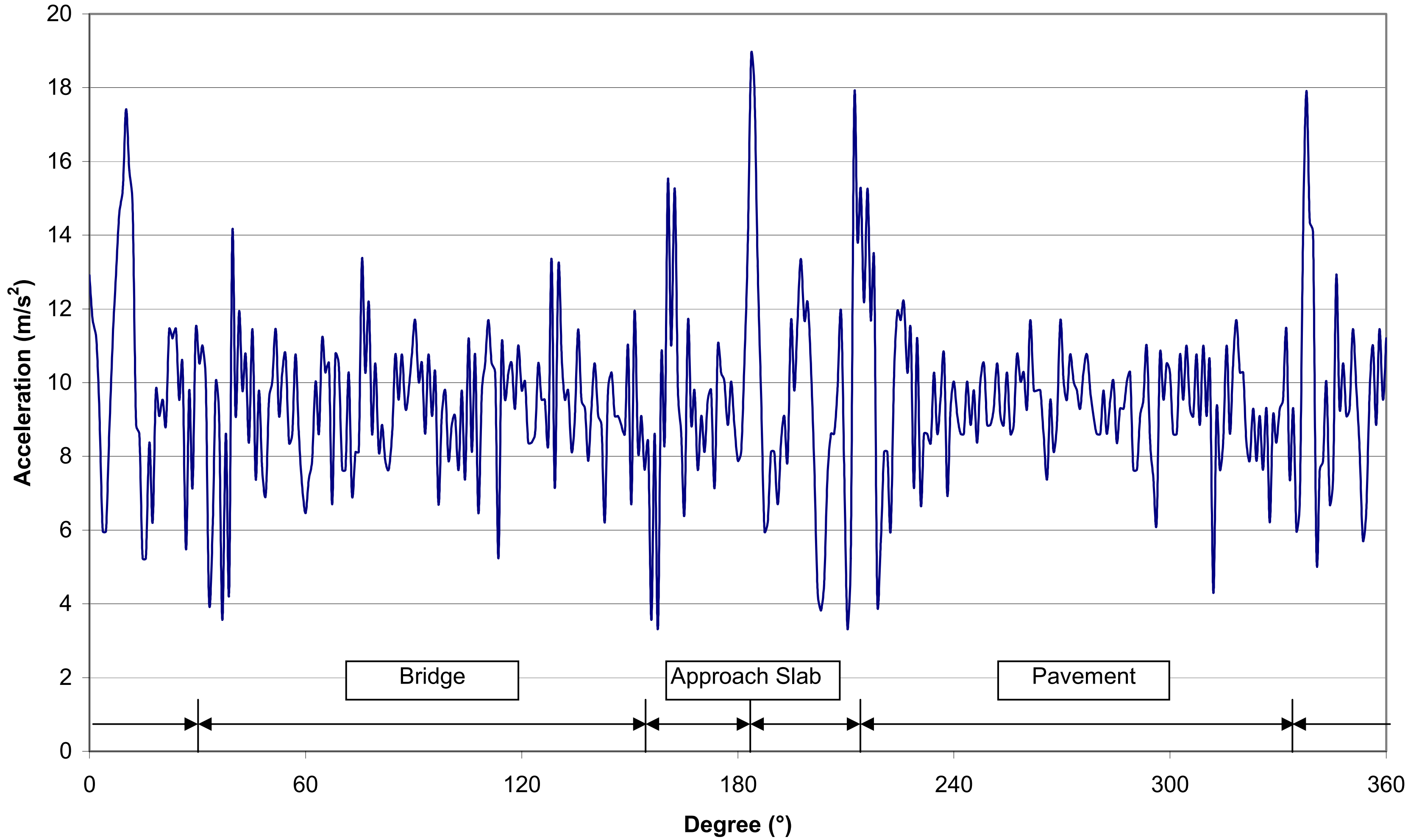


Figure I-9. Test 9 – Acceleration (Cycle No. 50,000).

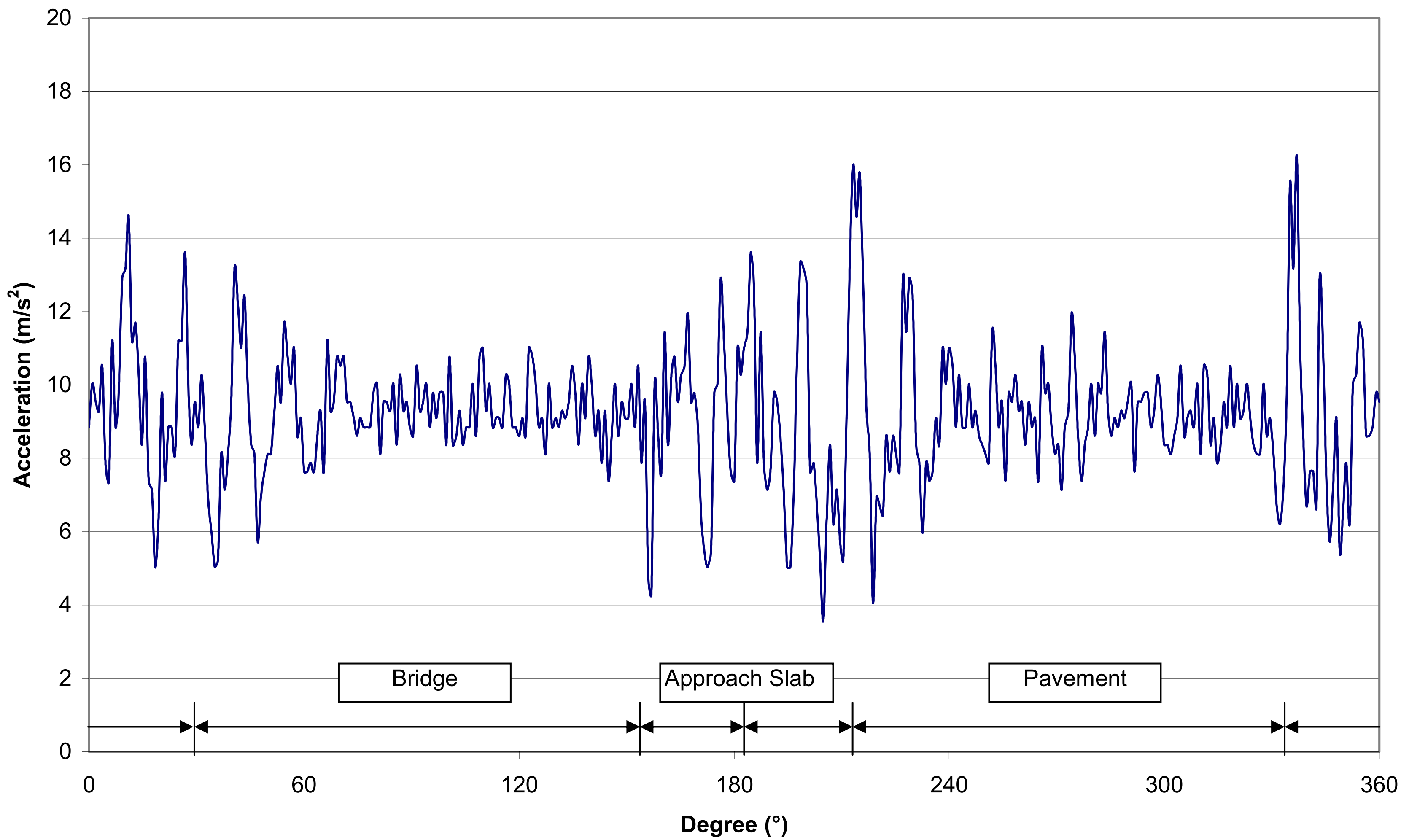


Figure I-10. Test 9 – Acceleration (Cycle No. 100,000).

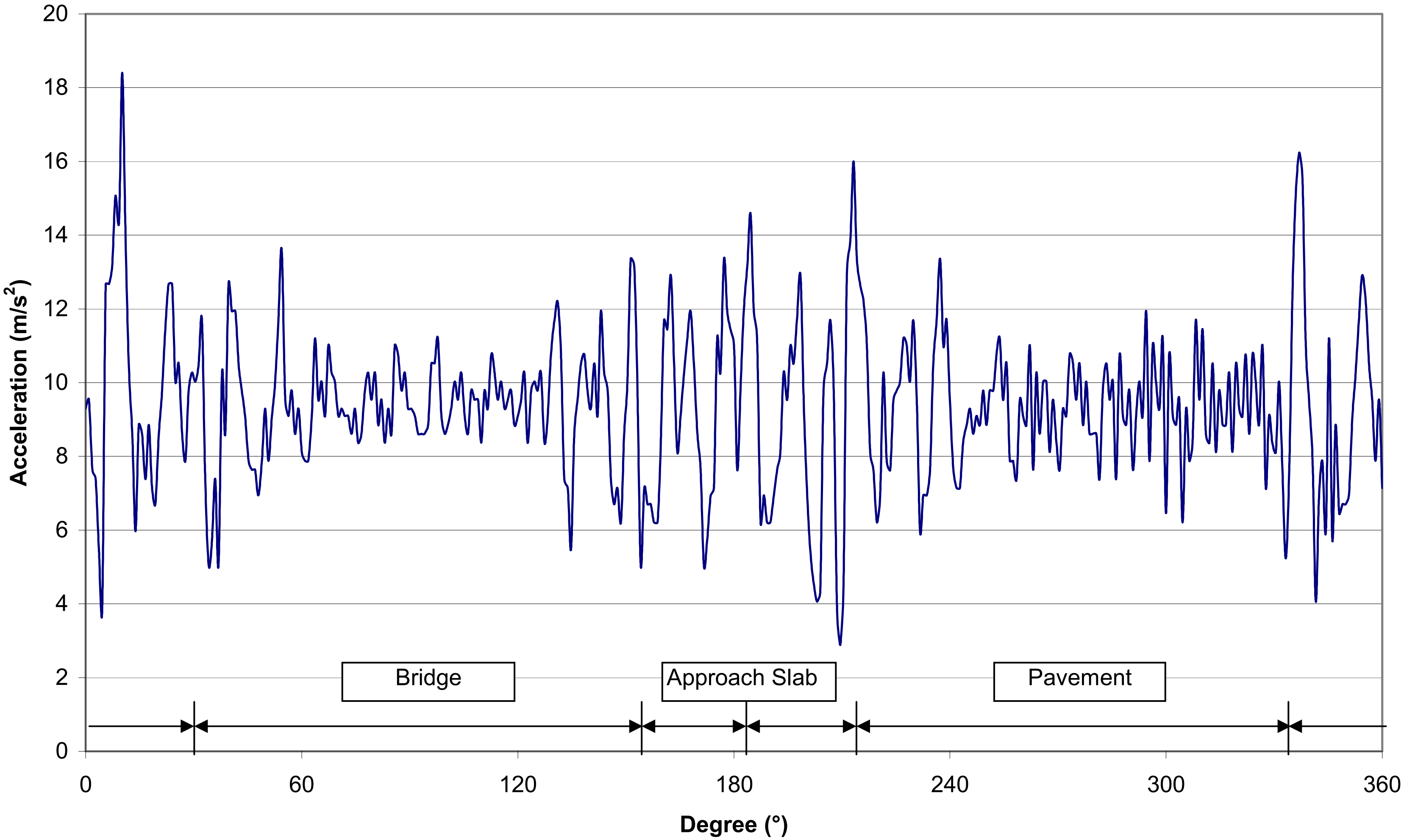


Figure I-11. Test 9 – Acceleration (Cycle No. 200,000).

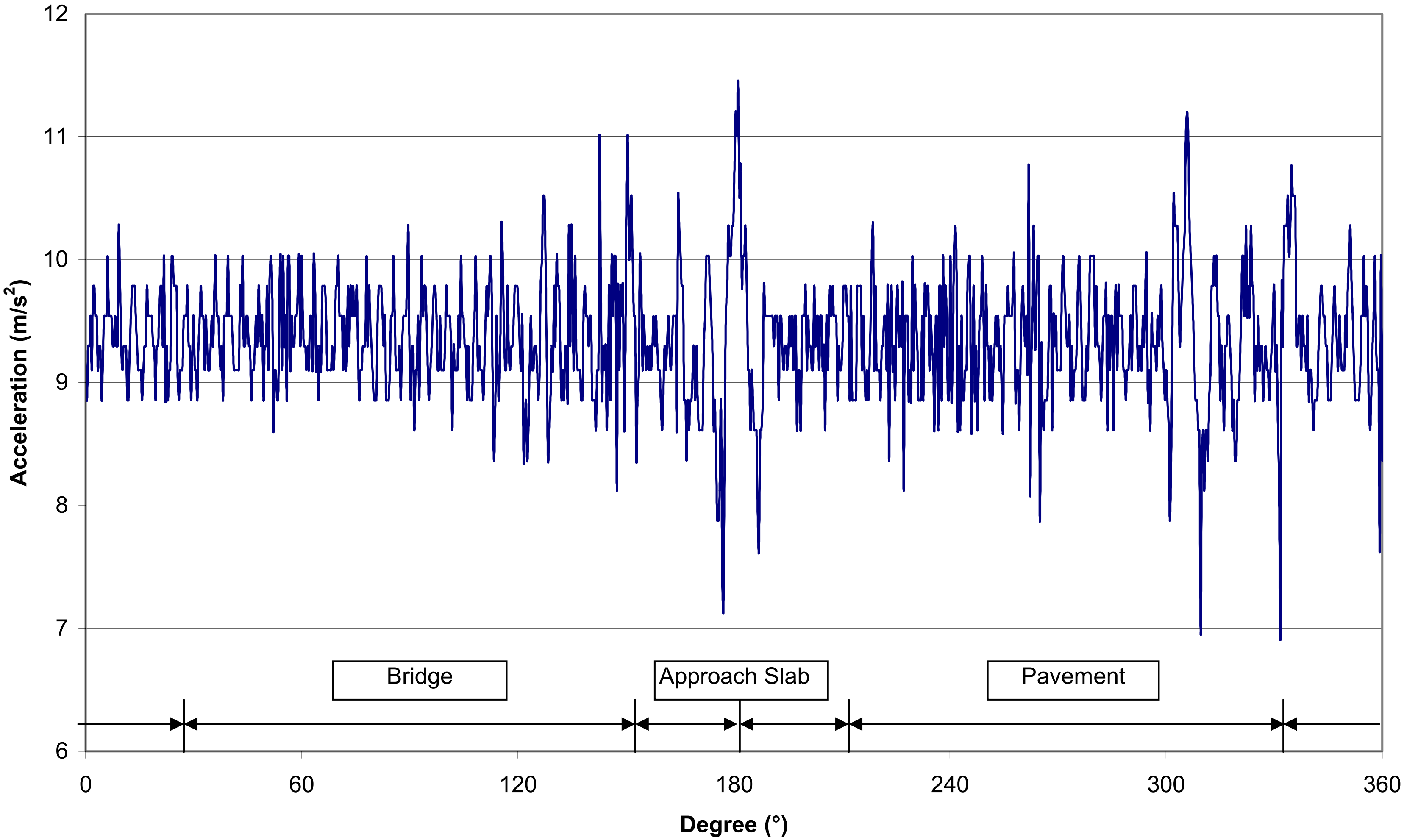


Figure I-12. Test 10 – Acceleration (Cycle No.1).

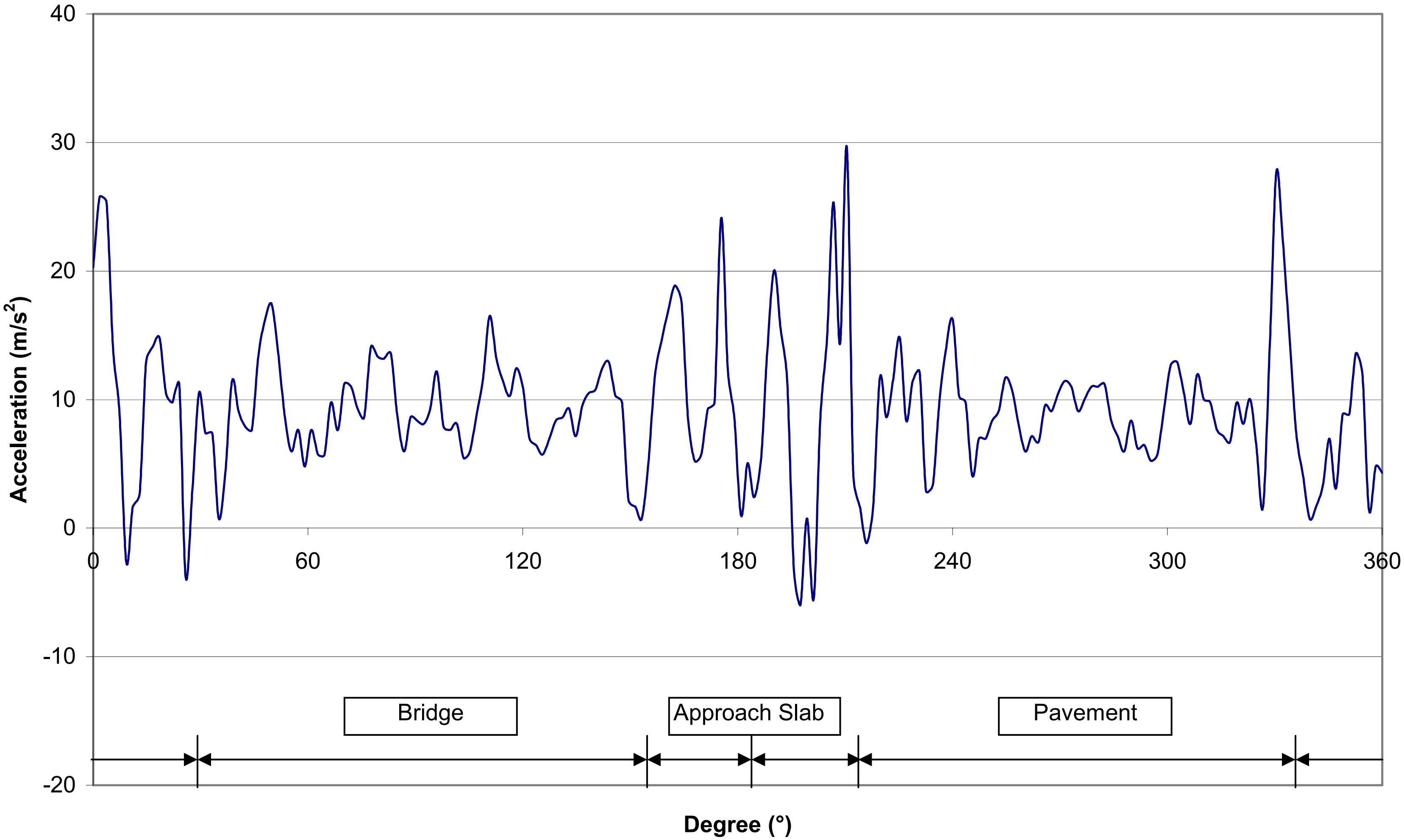


Figure I-13. Test 10 – Acceleration (Cycle No.2,500).

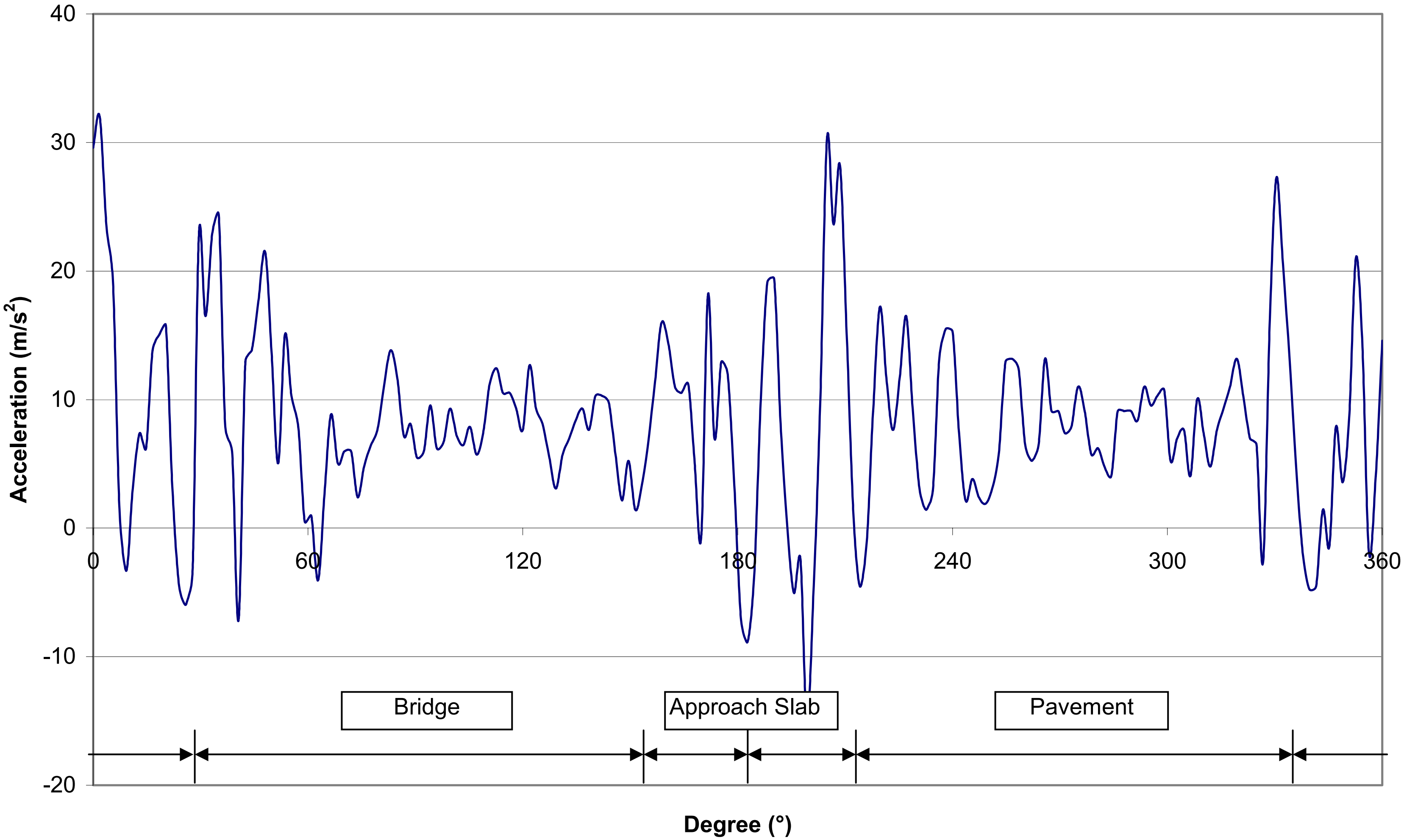


Figure I-14. Test 10 – Acceleration (Cycle No.5,000).