EVALUATION OF CONCRETE BARRIER AS ROCKFALL PROTECTION

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ABSTRACT

Rockfall is the movement of rocks down a slope which may be in the form of freefall, bouncing, rolling and sliding based on characteristics of slopes and nature of rocks. When rockfalls reach the roadway, they are hazardous to roadway users. Modified traffic concrete barriers that are standardized via crash test from the roadside as per National Cooperative Highway Research Program (NCHRP) Report 350 Test Level 3 (TL-3) criteria are commonly used by Ohio Department of Transportation (ODOT) to protect highways from rockfall hazards. The two most commonly used concrete barriers by ODOT are 32 inch high precast concrete barriers (PCB) and 42 inch high modified cast in place concrete barriers (CIP). The impact energy absorption limits and containment effectiveness of these barriers are relatively unknown and there is little knowledge of the efficiency of these barriers against rockfall. The objectives of this dissertation is to define the impact energy absorption limit and efficiency of these barriers against rockfall as well as the development of the new designs of concrete barriers for future applications.

Two phases of impact tests were conducted in the field with Test Rocks of reinforced concrete, steel balls and natural stone. Phase 1 impact testing was designed for a maximum impact energy of 70 kJ on test barriers. Phase 2 impact testing was designed based on results from Phase 1 for the new designs of concrete barriers (PCB and CIP) with an impact energy of more than 180 kJ. The revised designs for new concrete barriers include the use of smaller size reinforcing bars and spacing, steel/polypropylene fibers, use of welded wire fabrics (WWF) and black reinforcing bars without epoxy coating. The
results from Phase 1 impact testing indicated that current ODOT standard PCBs has a limit of an energy absorption capacity of 24 kJ under single impact and the capacity is much lower under multiple impacts. The modified traffic CIP concrete barriers has the maximum energy absorption capacity of 56 kJ under single impacts. During Phase 2 impact testing, the revised designs of PCBs with addition of fibers has the impact energy absorption limit of 45 kJ and reduced cracks patterns were observed. The maximum energy absorptions of the CIP concrete barriers with revised designs were observed to be as high as 156 kJ. The used of fibers increased the energy absorption capacities and also reduced the crack patterns in the concrete barriers. The use of steel fibers in the wet concrete increased the energy absorption capacity by around 40% and the polypropylene fibers increased the energy absorption capacity by 20%. The epoxy coating on the surface of the reinforcing bars reduced the bond to the surrounding concrete and increased the spalling of the concrete. The finite element modeling for the displacements and the strains were validated with high speed video of field data analyzed by ARAMIS.
DEDICATION

I would like to dedicate this dissertation to my advisor Dr. Anil Patnaik and my parents.
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CHAPTER I
INTRODUCTION

1.1 Introduction

Failures of earth slopes are common throughout the state of Ohio and can include both soils and rockfalls. Slope failures can be generally referred to as “mass movement” down the slope and can be classified into several categories. These categories can include earthflows, slumps, and rockfalls (Van De Grift and Sack, 2004). Rockfall movement may occur by free fall, toppling, bouncing, rolling, flowing, spread, sliding, or creeping. The Ohio Department of Transportation (ODOT Manual for Rockfall Inventory, 2013), defines Rockfall as “down-slope gravitational movement of material that is comprised of at least 51 percent rock.” “Rock is defined in the manual as any material found along a slope that when freshly exposed has the characteristics of in-place bedrock. Bedrock includes, but is not limited to, sandstone, siltstone, shale, limestone, dolomite, coal, claystone, and conglomerate.”

In Ohio, cut slopes in rocks can be divided into three broad types based on their mineral composition, grain size, and engineering characteristics. These are: i) Slopes composed mostly of competent rock units; ii) Slopes composed mostly of incompetent rock units; and iii) Slopes composed of interlayered competent and incompetent rock units (Admassu, 2010). The frequency and size of rockfalls depend upon joint spacing within the competent unit and the extent by which it has been undercut. Undercutting-induced
failures can be quite hazardous because of their instantaneous occurrence, high speed, and occasionally large volume of rocks involved. Another important aspect of undercutting in relation to slope movement is that undercutting is a dynamic process that continually changes the slope conditions. Therefore, both the amount of undercutting and the rate of undercutting can be important considerations while evaluating stability of rock slopes in Ohio (Admassu, et. al, 2012, Shakoor and Rodgers, 1992). As mentioned by Shakoor and Adamasu (2010), the common types of failures in Ohio slopes are planar failure, wedge failure, toppling, erosion and mudflows.

Most rockfalls in Ohio that result in significant damage and/or injury involve massive beds of sandstone or limestone (Hansen, 1995). Generally, rockfalls that result from these beds involve undercutting of the more competent beds by underlying weaker less competent beds and/or by the presence of joints, fractures and bedding planes within these formations. Rockfall potential is severe where these failure surfaces exist within the rock surface and/or where groundwater seeps into these openings and freezes. Frequent freeze-thaw cycles that occur during the colder months of the year can result in the separation of rock from the slope surface. In addition, rockfall can occur from the increase in rock mass due to saturation of porous formations.

The potential for rockfalls is higher in the eastern and southern portions of Ohio, which are characterized by steep slopes of several hundred feet. The rocks most prone to mass movement in eastern Ohio are red mudstones ("red beds") of Pennsylvanian and Permian age. These rocks are easily weathered and tend to lose strength when they become wet, forming rotational slumps or earth flows. Regardless of the type or location, any rockfall has a potential to cause injuries and fatalities to motorists or damage to the

The hazard from a rockfall is due to the suddenness of the occurrence and its terminal velocity, where debris can reach a velocity of 65 to 130 feet (20 to 40 meters) per second for large slopes. The block fragmentation during the impact of the rock with a bench or the slope on its way down or an impact with any other obstruction (such as a barrier at the base) is hazardous. Rockfall can cause serious (occasionally fatal) incidents on the roadway when rocks hit vehicles, vehicles swerve to avoid rocks and either run off the road or hit another vehicle, or vehicles hit rocks that have fallen onto the road. State departments of transportation (DOTs) have the following objectives for rockfall characterization, prevention, control, and containment in order to prevent impairment of traffic flow (Caltrans, 2009):

(a) Improve public safety, (b) Increase mobility and (c) Reduce tort liability exposure.

A number of different types of rockfall protection systems are currently used throughout the world (Yoshida et al., 2007; Badger et al., 2009; and Volkwein et al., 2011). One such barrier system involves the use of concrete barriers.

1.2 Rockfall Concrete Barriers

Placement of precast concrete barriers (PCBs) next to the roadway or construction of cast-in-place (CIP) concrete barriers along the edge of the pavement at the shoulder of the road are two common solutions practiced in most states including Ohio to protect roadway users from falling rocks. The outer geometry (shape and dimensions) of these barriers is standardized by crash-testing from the road side of the barriers in accordance
with the Test Level-3 (TL-3) crash test criteria described in National Cooperative Highway Research Program (NCHRP) Report 350 (Ross et al., 1993).

Rockfall concrete barriers are classified as rigid barriers. They absorb most of the impact and all of the residual kinetic energy of the falling rock instead of dissipating it as flexible nets do (Badger et al., 2009). Experience has shown that rigid walls have a tendency to break under high-impact loads (as seen in Figure 1.1), and shatter, sometimes violently (e.g., Badger et al., 2009). Because of their relatively small size, these barriers cannot contain large-sized rocks or high-energy rockfalls. Concrete barriers are generally believed to be suitable for rockfall protection where the resulting impact energy is in the range of 60 kJ to 100 kJ (Descoeurdes et al., 1999) or where catchment ditch effectiveness needs improvement.

![Figure 1.1 Failure of a precast concrete barrier under the impact from a rockfall](image)

Concrete barriers are also used to augment catchment ditches. A well-designed ditch can absorb a significant amount of energy. Therefore, the residual energy remaining during the rockfall is likely to be low if the falling rock engages the ditch before impacting with the barrier during the rockfall. Higher energy impacts that result from a rockfall
bypassing the ditch and impacting the barrier directly will impart all of its energy to the barrier. In such cases, concrete barriers are susceptible to severe damage and/or failure arising from the impact of a single rock or multiple rock impacts (Figure 1.1). Local and global failures of concrete barriers can occur if the energy of a rockfall event exceeds the structural capacity of the barrier to resist impact. Therefore, small sized concrete barriers will not stop large or high-energy rockfalls.

1.3 Current ODOT Practice

For protection against rockfalls, ODOT mostly uses D-50 (or D-42) CIP concrete barriers or D-32 PCBs that pass NCHRP 350 TL-3 requirements from the road side of these barriers. Typical dimensions of such concrete barriers are shown in Figure 1.2. These details were extracted from a recent ODOT project where D-42 CIP concrete barriers with #5 epoxy coated bars provided on both faces of the barrier were used. ODOT sometimes uses a taller barrier that stands at a height of 50 inches with dimensions similar to those shown in Figure 1.2.

![Figure 1.2](image)

**Figure 1.2** Some of ODOT rockfall concrete barriers options: (a) 32-inch PCB, (b) 42-inch CIP D-42 and (c) 50-inch CIP concrete barrier D-50
ODOT’s standard PCB is 32 inches in height (details are shown in ODOT standard drawing RM4.2). Specifications are available for a 50-inch-high PCB as shown in ODOT standard drawing RM4.1. Concrete barriers are also used by ODOT as catchment elements if there are no existing catchments or if the existing catchments are found to be ineffective or too narrow. CIP concrete barriers are a viable option because existing concrete slip form equipment is used to create these barriers, and local contractors are familiar with installing these barrier systems.

The use of these common barriers in Ohio appears to be an economical and practical solution in lieu of acquiring significant amounts of property for right-of-way, laying back rock slopes, or widening/deepening ditches. Concrete barriers are currently reported to cost between $62 and $144 per lineal foot for installation. In addition to the length of a barrier, cost is also dictated by the height of a barrier.

1.4 Problem Statement

While placement of PCBs or 42” (or 50”) high CIP concrete barriers along the edge of the pavement is a common solution practiced in Ohio to protect roadway users from falling rocks, the containment effectiveness of these barriers is relatively unknown for rockfall events. PCB and CIP concrete barriers were developed by crash-testing from the road side of the barriers without consideration of the impact of rocks from the ditch side for rockfall applications. ODOT’s goal is to achieve a rockfall catchment effectiveness of 95%. However, it appears that rockfall concrete barriers have been put in practice with little to no engineering information supporting its use or documented verification of their catchment effectiveness. An evaluation of the current designs and any potential improvements to the standard PCB and modified CIP concrete barriers systems will
quantitatively determine the catchment effectiveness and also ensure that ODOT barrier designs are sufficient to provide protection from rockfalls to achieve the 95% rockfall catchment target.

The primary focus of this dissertation was to simulate rockfall impacts on ODOT modified CIP concrete barriers to determine the efficiency and adequacy of these barriers through full-scale impact tests leading to quantitative definition of the energy absorption capacity and limitations of concrete barriers. An additional goal was to refine and enhance the impact energy absorption of concrete barriers, investigate the use of suitable materials and identify the limitations of the applicability of concrete barriers with modified details and designs.

This dissertation is also aimed at providing the required background and design basis to ODOT in the development of a technical guidance document. This guidance can include a basis for energy absorption due to rockfall and related protection levels, as well as rockfall debris containment by concrete barriers so that the debris does not spill over to the adjacent roadway. Research results would quantify the energy absorption capacity of the currently used 50 or 42-inch CIP barriers (or 32-inch PCB barriers) based on acceptable confidence levels from statistical analyses. Furthermore, the energy absorption capacity of the rockfall concrete barriers is to be improved by making suitable modifications to materials and reinforcement details.

1.5 Objectives
The overall objectives of this dissertation which includes the field impact tests in two phases with data analysis from strain gages, accelerometers and high speed camera as well as the finite element modeling are summarized as follows:
• Quantify the limitations of concrete barriers by establishing strength and energy absorption limits through impact tests and rollout tests, primarily for PCB 32” and CIP 42” or 50” concrete barriers;

• Develop alternative design approaches and/or details to improve the energy absorption capacity of these concrete barriers and reduce the possibility of local shattering by using alternative methods;

• Provide a basis for potential revisions to ODOT practices.

1.6 Scope of this Dissertation

This dissertation provides details of impact tests conducted in the field on ODOT standard PCB and CIP concrete barriers, and revised PCBs and CIPs concrete barriers. The dissertation is organized into 7 Chapters. Chapter 1 deals with the introduction and problem statement. Chapter 2 include a summary of the literature review. Chapter 3 describes the Phase 1 impact testing methodology and results. Chapter 4 describes Phase 2 impact testing and results. Chapter 5 presents an analytical method for the determination of the absorbed energy by CIP concrete barriers. Chapter 6 describes validation of the finite element modeling with ARAMIS analysis. Chapter 7 summarizes the conclusions and future recommendation.
CHAPTER II
LITERATURE REVIEW

2.1 Introduction

Reinforced concrete is one of the most widely used construction materials worldwide. In many circumstances, the reinforced concrete structures are subjected to static or dynamic loads based on the duration of the loads with respect to the natural period of structures and the mechanical properties of reinforced concrete varies according to the loading rate. The compressive and tensile strength of the concrete and reinforcing bars increases significantly under the dynamic loads due to higher strain rates. Typical strain rates for dynamic loads are shown in Figure 2.1. Hence, the performance of reinforced concrete under impact loading is different when compared to that under static loading.

![Strain rates associated with different types of loading](image)

**Figure 2.1** Strain rates associated with different types of loading (Ngo, et al., 2007).

As seen from the figure, impact loads generally have strain rates in the range of 1 sec\(^{-1}\) to 100 sec\(^{-1}\). Very high strain rates are produced by blast loads with range of 100 sec\(^{-1}\) to 10,000 sec\(^{-1}\).
2.2 Impact Load on Reinforced Concrete Structures

Figure 2.2 shows an idealized impact loading profile. The impact loads are characterized by steep rise to the peak in the force applied to the structures and decrease of forces in short period of time. The large-scale tests and computer simulations are usually employed in the investigation of the structures under impact loads due to complex interaction phenomenon during collision.

![Idealized impact loading profile](image)

Figure 2.2  Idealized impact loading profile

2.2.1 Compressive Behavior of Concrete

The mechanical properties of the concrete are influenced by the strain rate of load applied. Higher strain rates result in higher load carrying capacity of the concrete. Euro-International Committee for Concrete-International Federation for Prestressing (CEB-FIP) Model Code 1990 formulation gives the most comprehensive model for strain rate enhancement of concrete both in tension and compression.

Figure 2.3 on the next page shows the stress-strain relationship for different strain rates. As the stress-strain curves are linear, the stiffness in the dynamic and static loading case does not vary significantly from each other. Under dynamic loads, increment up to 4...
times the static values in compression and up to 6 times static values in tensions for the strain rates in the range of $10^2$ to $10^3$ per second have been reported (Grote, et al., 2001).

2.2.2 Tensile Behavior of Concrete

The tensile strength of the reinforced concrete is higher under higher values of strain rates of load application. There is an increase in tensile strength of 13 times under quasi-static strength with rate of more than 100 sec$^{-1}$. There is more uniformity in tensile behaviors of concrete under high strain rates in comparison to those under compression. In addition, the higher strain rates have a higher impact on tensile strength than compression strength. However, with strain rates lower than 10 sec$^{-1}$ there is no difference in the behavior of concrete in tension and compression (Pajak, 2011).

![Stress-strain curves of concrete at different strain rates](image)

Figure 2.3 Stress-strain curves of concrete at different strain rates (Ngo, et al., 2004)
2.2.3 Behavior of Reinforcing Bar under Impact Loading

Higher strain rates of impact loads alters the strength of steel in reinforced concrete structures. The mechanical behaviors of steel reinforcing bars can increase due to high strain-rate effect. Both the yield and ultimate stresses of rebars increase under higher strain rates when compared to the static values. Under impact loading, the 60% increment in mechanical strength of rebars are known for strain rates up to 10 sec\(^{-1}\) and up to 100% for strain rates around 225 sec\(^{-1}\) (Malver and Crawford, 1998). Based on literature review, a limited amount of research has been performed on the effects of strain rate on the rebars-to-concrete bond strength and the effects are not well-defined. The use of epoxy coated bars in reinforced concrete add to the uncertainty with regard to the bond-strength of reinforced concrete under dynamic loading.

2.3 Failure Mode of Reinforced Concrete under Impact Loads

The behavior of concrete barriers under impact/blast load has been studied in a limited manner in the past (El-Salakawy, et al, 2002; Murthy, et al, 2010; Coughlin, et al, 2010; Wilt and Chowdhury, 2011). There is no well-defined unified approach for determining the failure mode or the resistance of reinforced concrete under dynamic loading.

The impacts on the structures may be in the form of hard impacts and soft impacts. In hard impact, the impacting object shows little damage compared to the structures, whereas in soft impact the impacting object gets significant damages. The effect of impacts on reinforced concrete are summarized as: i) the localized failure mechanisms and most energy absorption is mainly local deformations. ii) Impact energy absorbed due to deformations in bending and/or shear energy in global structures or iii) combination of
local failure mechanisms and global failure mechanisms (Hrynyk, 2013). Figure 2.4 shows the failure mechanisms due to impact loads.

Field impact tests are used to define the failure mode of the reinforced concrete structures. For concrete barriers, several types of impact tests were reported by others, such as bogie impact tests, pendulum impact tests and vertical drop tests. The properties of the reinforced concrete structures under impact loading need further research to understand impact phenomenon. This is useful to design structures for rockfall impact, vehicular crash, and blast loading.

Figure 2.4  Missile impact effects on concrete target: a) penetration, b) cone cracking, c) spalling, d) cracks on: i) proximal face and ii) distal face, e) scabbing, f) perforation, and g) overall target structure response (Murthy, et al., 2010)

Computer simulation can be used to validate the results from the full-scale experimental impact tests. LS-DYNA (developed by Livermore Software Technology
Corp., Livermore, Ca.) is an explicit finite element modeling application used for the analysis of highly transient dynamic problems (LS-DYNA 971 Manual). ABAQUS is another explicit finite element modeling software application (Abaqus 6.11 User’s Manual) which is also suitable for such analyses.

Field impact tests are used to define the failure mode of reinforced concrete structures. For concrete barriers, several types of impact tests were reported by others, such as bogie impact tests, pendulum impact tests and vertical drop tests as described in the following sections.

2.4 Bogie Impact Tests

A bogie impact tests have been used by Federal Highway Administration (FHWA) to simulate the dynamic impact analysis of highway appurtenances such as concrete barriers in crash tests. A bogie is configured with different noses to simulate different crash characteristics, and an accelerometer is fixed at an approximate location of the center of gravity. It is guided by an external driving mechanism and made to accelerate towards the barrier for impact tests.

Abu-Odeh (2008) used a 5000-lb (2268-kg) bogie impactor to investigate the impact resistance of the bridge rail design as shown in Figure 2.5. The bogie was fitted with a three-cylinder steel crushable node, and the tests were conducted with nominal speeds of 15 mph (24 kph) and 20 mph (32 kph).

In that study, the first bogie impact test was conducted at the speed of 15 mph (24 kph) toward the concrete barrier delivering impact energy. The concrete barrier showed no fracture as the barrier absorbed energy to that level. The impact energy is dissipated through crack patterns that developed during the impact. The second test was conducted at
the higher bogie speed of 20 mph (32 kph) with higher impact energy delivered to the concrete barriers and in failure mode of the barrier is shown in Figure 2.5.

![Figure 2.5 Bogie impact testing for the T501 safety shape rail (a) test setup, (b) failure of barriers after tests (Abu-Odeh, 2008)](image)

2.5 Pendulum Impact Tests

The pendulum impact test is one of the most commonly used methods for estimating the performance of structures under impact loading. Several experiments have been performed in different countries by several researchers (Bank, et al. 1998; El-Salakawy, et al., 2002; Mitchell, et al., 2006; Aminata, et al., 2008; and Ahmed, E. A., et al., 2013). In this test, the impacting mass is released from the desired height to impact the test barrier supported on a horizontal surface. The conservation of total energy for the falling ball (neglecting minor losses) is applied in order to calculate the impact velocity of the mass as follows:

\[
mgh = \frac{1}{2}mv^2
\]

(2.1)

Where \(m\) is the mass of the impacting rock, \(g\) is the gravitational acceleration, and \(v\) is the velocity of impact.
The energy absorbed by the structure is calculated from the energy equation based on pendulum motion of the impacting mass assuming negligible losses in the energy. The impulse driven to the structures by the impacting object is computed by using the velocity during impact and the impact duration.

\[
F_{\text{impact}} = m \frac{\Delta V}{\Delta t}
\]  

(2.2)

Where \(F_{\text{impact}}\) is impact force, \(\Delta v\) is the change in velocity, and \(\Delta t\) is the change in time.

El-Salakawy, et al., (2002) investigated the resistance of the Canadian highway bridge concrete barriers, namely, PL-2 and PL-3 at the department of civil engineering, Université de Sherbrooke, (Quebec, Canada) as shown in Figure 2.6. The experiment was conducted both under static loading conditions and pendulum impact tests.

Figure 2.6  Photo for the test set-up (El-Salakawy, et al., 2002)
The 6614-lb (3000-kg) pear-shaped iron ball was used in the experiments as shown in the figure. The impacting balls were raised to the desired height by using crane and were released to impact the barrier. Strain gages measured strains at critical locations in the reinforcing bar, and accelerometers mounted on the back face of the wall measured the peak accelerations and durations of the impact load.

The impacting energy delivered by the falling ball from height of 3.2 m (10.5 ft) for PL-2 barriers and 3.5 m (11.5 ft) for PL-3 barriers were 87 kJ and 101 kJ, respectively. The impact velocity of the ball was calculated using Equation 2.1. The velocity of the ball was reduced to zero after the impact.

Stresses in the concrete barriers are due to forces from impacting balls and the resulting inertia force induced after impact. Only the inertia force in due to vibration of concrete barriers exists after the duration of the impact. Experimental results include crack patterns on the barrier due to impact loading. Figure 2.7 shows the crack pattern on the concrete barrier due to the large stresses from the impacting ball.

Figure 2.7 Crack patterns observed during the test (El-Salakawy, et al., 2002)
A summary of the pendulum impact test results performed by El-Salakawy, et al., (2002) are shown in Table 2.1. The accelerations from the same falling height are different due to the non-uniform torque applied to tighten the base slab of the barrier to the test bed. The duration of the impact is around 0.1 sec with the peak value in the first 0.03 sec.

Table 2.1 Summary of impact test results from El-Salakawy, et al., (2002)

<table>
<thead>
<tr>
<th>Barrier</th>
<th>Acceleration (m/s²)</th>
<th>Impact load (kN)</th>
<th>Max. measured crack width (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Front face</td>
</tr>
<tr>
<td>PL-2 series</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PL2-ST1</td>
<td>257</td>
<td>750</td>
<td>0.65</td>
</tr>
<tr>
<td>PL2-ST2</td>
<td>234</td>
<td>690</td>
<td>0.45</td>
</tr>
<tr>
<td>PL2-IS1</td>
<td>248</td>
<td>731</td>
<td>0.75</td>
</tr>
<tr>
<td>PL2-LS2</td>
<td>245</td>
<td>728</td>
<td>0.85</td>
</tr>
<tr>
<td>PL-3 series</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PL3-ST1</td>
<td>243</td>
<td>716</td>
<td>0.60</td>
</tr>
<tr>
<td>PL3-ST2</td>
<td>173</td>
<td>511</td>
<td>0.56</td>
</tr>
<tr>
<td>PL3-IS1</td>
<td>252</td>
<td>744</td>
<td>0.54</td>
</tr>
<tr>
<td>PL3-IS2</td>
<td>220</td>
<td>649</td>
<td>0.80</td>
</tr>
</tbody>
</table>

The Figure 2.8 shows the pendulum impact setup developed by Mitchel, et al., (2006) at The University of Texas at Austin. It has a 22-ft (6.7-m) tall steel frame and four cables suspending a 855 kg (1885 lb) mass. The swinging mass can be released from the desired height with a maximum lifting height of 16 ft (4.9 m).
The impulse produced from this pendulum impact tests are required to be similar to be the actual vehicle impact. Therefore, test results in NCHRP Report 350 TL-3 crash test conducted to measure the structural behavior of a 1997 Geo Metro colliding with a Texas Department of Transportation (TxDOT) T77 steel bridge barrier (TTI 2002) at the Texas Transportation Institute (TTI) in August 2002 by Bullard, et al. (2006) was used for verification. The transverse acceleration (normal to the plane of the longitudinal barrier) of the impact of Geo Metro against the steel bridge barrier is shown in Figure 2.9.

Figure 2.9   Metro NCHRP Report 350 TL-3 crash test (a) acceleration history and (b) photograph of impact with steel bridge barrier (Bullard, et al., 2002)

The results from impact tests matched values obtained from the surrogate vehicle reasonably well as shown in Table 2.2.

Table 2.2   Comparison made between impact pendulum test setup and the 1997 Geo Metro for NCHRP Report 350 TL-3 (Mitchell, et al., 2006)

<table>
<thead>
<tr>
<th>Experimental test</th>
<th>Result from tests</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Impact energy (kJ)</td>
</tr>
<tr>
<td>1997 Geo Metro</td>
<td>38.5</td>
</tr>
<tr>
<td>Pendulum Impact Test on Steel Barrier</td>
<td>38.5</td>
</tr>
</tbody>
</table>
2.6 Vehicular crash tests

The Ohio Department of Transportation (ODOT) uses D-50 (or 42") cast-in-place (CIP) and D-32 portable concrete barrier (PCB) that pass National Highway Research Program (NCHRP) Report 350 Test Level 3 (TL-3). A report on full-scale crash tests conducted on the ODOT 32-inch New Jersey Shape PCB is also available (MacDonald and Kirk, 2001). The test was conducted at the Transportation Research Center (TRC) in East Liberty, Ohio in the Fall of 2001 under the impact conditions corresponding to NCHRP Report 350 (Test 3-11), in which a 4400-lb (2000-kg) 3/4-ton pickup truck impacts the barrier at 62.2 mph (100 km/hr) at an angle of 25°. If the test vehicle did not penetrate the area on the other side of the barrier, the test was deemed passed. Figure 2.10 shows the design vehicle impacting the barrier.

Figure 2.10  Test vehicle prior to Crash Test 3, showing angle of impact (MacDonald and Kirk, 2001)
2.7 Impact Tests on New Rockfall Barriers in Japan

Aminata, et al. (2008) proposed new protection against rockfalls using ductile cast iron panels that provide a simple, long-lasting and low-cost structure with maximum impact energy dissipation. The full-scale field tests were conducted to evaluate the performance of different rockfall protection and to enhance their mechanical behavior during rockfall impact. This study also aimed to improve the design and repairing methods of such systems. A ductile cast iron panel of 500 mm (1’-8”) in height and 1 m (3’-3”) in length was used in the study.

In order to simulate rockfall impact, various energies of falling rocks (fabricated) were assumed and made to collide with full-scale ductile cast iron panels. During first field tests, three test rocks were rolled down a 17 meter (55.8 feet) high slope with an angle of inclination of 45° (Figures 2.11a and 2.11c) made to collide with the ductile cast iron panel frames several times. The test rocks has diameters: 150mm (6 inch), 300mm (12 inch), 600 mm (24 mm) and weight: 0.36 kN (81 lb), 1.67 kN (375 lb), and 5.0 kN (1124 lb) respectively.

Additional free fall collision tests using two were conducted (Figures 2.11b and 2.11d) due to difficulty in controlling forces due to friction between the mass and rough sloping ground. A mass of 5.0 kN (1124 lb) was lifted to a height of 11 meters (37 ft), and then dropped. The impact forces, energies, velocities and displacements were computed from the acceleration records obtained from a 3-dimensional accelerometer installed in the falling rock as follows:

Velocity (v) is obtained by the time integration (dt) of acceleration (a) using the following equation:
\[ v = \int a \times dt \]

Figure 2.11  New protection walls against rockfall using a ductile cast iron panel: a) Schematic of the first field test, b) Schematic of the second field test, c) Backhoe dropping a rock, d) Experimental condition during the 2\textsuperscript{nd} test (Aminata, et al., 2008)

Displacement \( u \) of the falling rock can be obtained by the time integration of velocity \( v \) using the following equation:

\[ u = \int v \times dt \]
The impact force (F) is calculated by the product of acceleration (a) and mass (m):

\[ F = m \times a \]

Absorbed energy of protection wall (\( E_a \)), is calculated by the time integration of the product of impact force (F) and incremental displacement (u):

\[ E_a = \int F \times \Delta u \times dt \]

The results of the full-scale tests of the rockfall protection wall are shown in Figure 2.12. The acceleration of the falling rock peaks at about 0.01 second and decreases after that point (Figure 2.12a). The velocity peaks at the time of impact and decreases continuously (Figure 2.12b). The variation of the absorbed energy and impact force with time are shown in Figures 2.12c and 2.12d. Numerical simulations using finite element models with larger impact load were used to investigate stresses and strains inside the rockfall protection structures for larger impact energies. The material feature configurations and their excellent performance as energy absorbers were demonstrated.
2.8 Fiber Reinforced Concrete Barrier under Impact/Blast Loading

Fiber reinforced concrete is a type of concrete with fibers dispersed uniformly during mixing or casting. Different types of fibers can be used including steel, glass, synthetic, and basalt. Fiber reinforced concrete has increased tensile strength, cracking resistance, impact resistance, and shrinkage reduction compared to conventional concrete. The use of fiber reinforced concrete in comparison with conventional concrete increases the toughness or energy absorption of the concrete, resulting in reduced spalling of concrete under impact loading.

Coughlin, et al., (2010) investigated the blast resistance of a portable concrete barrier (PCB) to be used to increase the safety from explosives. The experiments were conducted at the Air Force Research Lab test range at Tyndall Air Force Base, Florida, as shown in Figure 2.13. The effect of fiber reinforced concrete in increasing the performance
of concrete barrier under blast load was discussed by presenting a comparison with the performance of traditional concrete under similar conditions.

Figure 2.13  Barriers before testing (Coughlin, et al., 2010)

In this study, four barriers constructed using different types of fiber reinforced concrete along with one constructed from normal concrete without fibers were tested. Two test barriers had synthetic fibers, two had steel-synthetic blend with different fiber content, and another barrier had traditional normal weight concrete which was used as control specimen. A summary of the barriers tested is shown in Table 2.3.

Table 2.3  Barriers test matrix (Coughlin, et al., 2010)

<table>
<thead>
<tr>
<th>Barrier Type</th>
<th>Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>K-1</td>
<td>Standard concrete (control)</td>
</tr>
<tr>
<td>CFRC</td>
<td>Carbon fiber</td>
</tr>
<tr>
<td>NFRC</td>
<td>Nylon Fiber</td>
</tr>
<tr>
<td>SS-H</td>
<td>Synthetic/steel fiber mix 1 (high fiber volume)</td>
</tr>
<tr>
<td>SS-L</td>
<td>Synthetic/steel fiber mix 2 (lower fiber volume)</td>
</tr>
</tbody>
</table>
Figures 2.14 and 2.15 show the experimental result from the blast load and the corresponding LS-DYNA simulations. These results indicate that the concrete barrier with carbon fiber shows less cracking and spalling of concrete when compared to that of traditional concrete barrier. As can be seen in these figures, the addition of fibers to concrete used for making barriers significantly enhances the performance of the barrier under impact loading due to the superior post-cracking behavior of reinforced concrete. Therefore, the concrete with fiber can dissipate impact energy better than traditional concrete.

Figure 2.14  Comparison of front (left) and back (right) crater mappings from analytical and experimental studies, barrier K-1 (Coughlin, et al., 2010)

Figure 2.15  Comparison of front (left) and back (right) crater mappings from analytical and experimental studies, barrier CFRC (Coughlin, et al., 2010)
Table 2.4 compares the experimental and analytical surface damage of the regular concrete barrier and carbon fiber reinforced concrete barrier studied by Coughlin, et al., (2010). The computer modeling of the damages surface of concrete helped in understanding effect of impact loading when there is no actual tests performed.

Table 2.4  Experimental and analytical comparison of surface damage (Coughlin, et al., 2010)

<table>
<thead>
<tr>
<th>Barrier</th>
<th>Barrier face</th>
<th>Surface damaged (%)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Experimental</td>
<td>Analytical</td>
</tr>
<tr>
<td>K-1</td>
<td>Front</td>
<td>22.2</td>
<td>38.5</td>
</tr>
<tr>
<td></td>
<td>Back</td>
<td>60.4</td>
<td>62.2</td>
</tr>
<tr>
<td>CFRC</td>
<td>Front</td>
<td>9.8</td>
<td>12.1</td>
</tr>
<tr>
<td></td>
<td>Back</td>
<td>29.8</td>
<td>22.7</td>
</tr>
</tbody>
</table>

It can be concluded that the normal concrete reinforced with reinforcing steel bars tends to become brittle or can fragment into pieces under impact loading due to very low tensile strength of concrete. The energy absorption capacity of concrete under impact loading can be increased significantly with the addition of fibers to the concrete mix. Fiber limit cracking of concrete and also increases the energy absorption capacity of the concrete under impact loading.

2.9 Rockfall Protection Measures

Rockfall protection can be managed either by active mitigation or passive mitigation. The active mitigation system was aimed to prevent detachment of the rock mass which may include rock bolting, slope retention systems, wire meshing, and shotcrete, as well as changing the prevalent conditions in the initiation zone by changing slope geometry, slope dewatering or re-vegetation. On the other hand, the passive mitigation system acts after the detachment of rock mass. Passive mitigation solutions include the use of drape nets, rockfall catchment fences, concrete barriers and embankment.
2.9.1 Modification of Slope Geometry

The design slopes angles for a given site is mainly influenced by the existing structural geology at a site. Structural geology includes discontinuities within rock mass such as joints, bedding planes, foliation, shears, and faults (Brawner, 1994). These analysis and experiences helps in designing a "safe" design slope angle for a given slope. The benches when created on the slopes sections or ditches at the bottom of the slope can be effective in reducing rockfall hazards. The computer simulations of rocks helps in locating the position of these benches. The benches can be covered with loose rock fill or earth material which absorb a large part of the impact energy. The ditches at the bottom of the slope are used to arrest the rock through impact energy dissipation and also used to change the rock movement from falling to rolling (Brawner, 1994). However, mid-slope benches are usually not accessible for cleanout and any accumulation of rockfall debris on the bench can serve as a "launching feature" for continued rockfall.

Within Ohio the prevailing discontinuities are joints and bedding planes. The bedding planes become a controlling discontinuity when the underlying bedrock strata is less durable than the overlying strata. As the less durable stratum weathers more rapidly than the overlying stratum, loose rock due to intersecting joint sets becomes unstable from the loss of basal support, resulting in rockfall over time.

2.9.2 Concrete Barriers as Rockfall Protection

Hazards associated with rockfall are present throughout the Ohio. However, roadway rock cuts and exposed bedrock slopes are more prevalent in the high relief terrain regions of the state. These areas are located to the east and south, and in areas where erosion has deeply dissected the ground surface. Topographical differences and the presence of
bedrock closer to the surface in these areas can result in a higher potential for rockfall hazards. Rockfalls are generally less common in portions of the state to the north of the historical glacial boundary. Through most of these areas, glacial ice action has leveled much of the earth surface and deposited soil over the rock surface. Roadway rock cut slopes and exposed bedrock in these portions of the state are less prevalent.

To address potential hazards associated with rockfall, ODOT developed Geotechnical Bulletin 3 (GB-3), “Rock Cut Slope and Catchment Design” (ODOT, 2011). This document provides guidance for the design of rock cut slopes and catchment to reduce rockfall hazards. The primary means of prevention of rockfall entering the roadway, “catchment”, presented in GB-3 is through the use of catchment ditches and clear zones. The document further indicates that barriers or other catchment systems are to be used as solutions to rockfall problems on existing slopes. GB-3 also allows the use of other catchment designs stating that “these items should be used in combination or independently only in special circumstances as part of a new slope design (for example to satisfy roadside grading criteria as discussed in the ODOT Location and Design Manual, Volume 1, Section 307.2.1, to address right-of-way concerns, to address changing slope condition of bedrock quality, or where the cut is very high).”

After the introduction of GB-3, it was determined that to meet the proposed 95% catchment criteria specified by ODOT, the design model required the acquisition of significant amounts of property for construction and right-of-way. To provide adequate catchment, up to 40 feet wide catchment ditches and clear zones would be required for slopes greater than 90 feet in height. In cases where the cut was through a hill, the catchment and clear zone could have a combined width of up to 80 feet. The wide
catchment ditches also required additional rock excavation and excavation/fill balancing or fill disposal. This in turn required the acquisition of additional right-of-way or disposal areas to dispose the excavated rock. While catchment ditches are the preferred design criteria in GB-3, consideration of other, and more economically practical catchment designs are considered to reduce the amount of right-of-way required for the remediation of existing slopes.

A review of existing state departments of transportation literature was conducted by the researchers to assess the current best practices for rockfall management and rockfall catchment construction. This included obtaining readily available information from all of the state DOTs. The information included geotechnical design/procedures manuals, construction design/specification manuals, rock cut slope design/guidance documents, rockfall/catchment design/guidance documents, and standard detailed design plans. Limited information was also collected from other sources that included presentations from engineering, manufacturing, and construction companies as well as government and academic research to identify the current best practices used for catchment enhancement through the use of rockfall barriers. The use of barrier systems on existing roadways has been a common practice throughout the United States in areas where existing right-of-way was insufficient and acquisition of additional right-of-way was impractical due to existing dwellings, structures, or roadways.

The rockfall barrier information review was further refined to include criteria involving rockfall barriers that are located between the toe of the rock face and the edge of pavement. There are essentially five types of barriers that are used for this. These include cast-in-place concrete barriers (CIPs), portable concrete barriers (PCBs), wall barriers,
Thrie beam guardrail barriers, and fencing barriers. Examples of each of the systems are presented in Figure 2.16.

![Cast-in-place (CIP) concrete barriers](image1)

![Temporary concrete barriers](image2)

![Thrie beam guardrail barriers](image3)

![Wall barriers](image4)

![Fencing barriers](image5)

Figure 2.16 Types of common rockfall barriers

There are other barrier systems that are also used. The type of systems deployed can range from very complex systems (in states with steep slopes that are several hundreds of feet in height and where rockfalls are common) to states where rockfalls from slopes are uncommon and there are no published design or protection criteria. ODOT disallows the consideration of the installation or the presence of guardrail along the road as a corrective action or method of catchment (ODOT Manual of Rockfall Inventory, 2013). This is because guardrails used by ODOT are not load rated to resist the impact of falling rock from the ditch side of the guardrail.
Images shown in Figures 2.17 and 2.18 demonstrate the limitations due to impacts from large boulders. In addition, to remain effective, concrete and other ridge barrier systems require continual maintenance. Slopes that produce talus and rockfalls will require clean out between the barrier and the slope to maintain the effectiveness of the catchment system. Concrete barrier systems present problems for snow removal and conveyance of stormwater on both sides of the barrier. These barriers also present a hazard to the motoring public, as the rigid barrier is a potential impact hazard. Therefore, the use of rigid concrete barriers should be weighed against the potential for a rockfall to enter the roadway. It is noted that use of deep catchment ditches beyond the shoulder may require some type of barrier as well.

It appears that ODOT will continue to use a “D-Type” profile on the traffic side of the concrete barrier. However, no specific profile has been specified for the ditch side of the barrier. One typical design for the ditch side is either a flat (vertical) profile or a profile with permissible 2 inch batter. ODOT has indicated that they will no longer install 32-inch high cast-in-place barriers. Instead, allowable design choices will include cast-in-place 42- and 50-inch barriers along the roadway shoulders. Cast-in-place barrier design specifications for ODOT use “QA/QC 1” concrete with 4,000 psi compressive strength, and some steel reinforcement (typically, #5 reinforcing bars with approximately 10 to 24 inch spacing both ways on both faces of the barrier). Review of the designs used by other states shows variations in profiles from an “F-Type”, a bowed wedge profile, single angle profile, and other similar variations of these profiles. Most designs use “Class C”, “Class A”, or “Class K” concrete, with a compressive strength between 2500 and 4500 psi and 5
to 7 percent entrained air. The barriers are also constructed with or without steel reinforcement depending on the particular DOT specifications.

Some state DOTs also recommend improvements to the catchment ditch area including the use of energy absorbing sand or stone that can reduce the energy and limit “bounces” and “rollouts” impacting a barrier. However, ODOT has determined that the stone or sand can become clogged with fines, organic materials or other debris that result in fouling of the catchment drainage. Therefore, these materials are not typically used in the catchment ditches by ODOT.

2.9.3 Catchment Ditches

One of the most effective rockfall protective systems for highways is the construction of a catchment ditch at the toe of the slope. Prior ditch design methods recommended that the base of a ditch be covered with a layer of gravel to absorb the energy of the falling rocks. A barrier may also be needed between the ditch and the roadway if the minimum width is not available to satisfy the design criteria adopted for a location or if the base of the ditch is not covered with a suitable energy absorbing material. Sometimes,
accumulation of debris at the base of the ditch can also reduce the effectiveness of the ditch suggesting the need for a barrier.

If a barrier is required, the location of the barrier may be determined theoretically by means of a rockfall analysis such as that used to calculate the trajectories of rockfall (e.g., CRSP). A barrier may be placed at a suitable distance from the toe of the slope so that at least 95% retention of rocks can be achieved. The kinetic energy of a falling rock is generally diminished because the rock first impacts the base of the catchment ditch, which is normally very soft to stiff and wet. However, the effectiveness of a ditch can diminish significantly if the ditch is not cleared periodically and is allowed to accumulate rock debris from previous rockfall events.

In selecting an appropriate catchment ditch dimensions with or without a barrier for a particular site, the following factors are considered:

- Rockfall sizes, falling heights and frequency of rockfalls;
- Geometry of slopes, and space availability at the base of the slope;
- Acceptable level of risk of rockfall damage;
- Availability of construction materials;
- Equipment availability; and
- Construction cost

Careful consideration of the above factors would lead to an acceptable and practical ditch design.

2.9.4 Other Types of Protection Barriers

Some of the protection barriers commonly used for rockfall protections includes flexible barriers and wire net systems, rockfall sheds and earth embankments
i) Flexible Barriers and Wire Net Systems

Flexible barriers are designed to absorb energy from falling rocks through large deformation of fence material and braking elements. The fencing material usually consists of deformable cables and/or mesh. Wire net can also be used with rail walls (relatively stiff) and other stiff barriers. The energy absorption capacity of such systems is rather low and is estimated to be about 10 to 50 kJ.

Flexible wire net systems, supported by hinged steel posts, were developed during the 1990’s (Descoeudres, et al., 1999). The energy absorbing capacity for these systems improved from about 250 kJ in the 1980s to more than 2000 kJ in 1990’s with the use of ring net construction. The newer systems comprise nets, supporting cables, wing cables, edge cables, brake elements, anchoring supports, and abrasion protection (Volkwein, 2014). Full-scale impact tests of such systems revealed that the impact energy absorption capacity of these systems can be up to 5000 kJ (Margreth and Rock, 2008).

However, very large deformations have to occur in order for these flexible barriers to be effective. Large deformations will need large areas in front of the fence to allow for the elongation of the system when rock strike occurs. ODOT’s experience is that they do not typically have the required room for these systems to be effective.

ii) Rock Sheds

Rock Sheds are used where there is frequent rockfalls and they are made from reinforced concrete structures having a roof covered by loose materials such as soil backfill for an additional energy absorbing material (Yoshida, et al. 2007). They are constructed at a slope having steep angle and where there is insufficient space at the toe of the slope to construct less expensive barriers. There is little maintenance with a good level of protection.
against rockfall accidents, compared to other rockfall barriers such as flexible nets or concrete barriers.

iii) Reinforced Earth Retaining Structures

Rockfall protection embankments have been used widely to stop high kinetic energy rockfall to protect the roads, or debris in civil engineering applications (Peila, et al., 2002). Earth embankments can absorb the kinetic energies of falling rocks, more than 30,000 kJ.

2.10 Impact Energy Absorption of Concrete Barrier as Rockfall Barriers

Rockfalls impairs traffic flow and can cause accidents leading to injury and fatality (Caltrans Workshop, 2009). There is very little evidence and experience of concrete barriers used as effective rock-fall protection. Guzzetti and Reinchenback (2005) found that there are two considerations required for evaluating the efficacy of rockfall concrete retaining structures (1) the maximum height of retaining structure can be smaller than the maximum height of the rockfall trajectories (2) the falling rock can have enough kinetic energy to break through a concrete barrier wall. Concrete barriers can normally contain rock-falls involving small rocks from small heights. Slopes with large surface roughness and slopes with low angles will likely lead to a decrease in velocity and bounce height. In such cases, the energy generated by the velocities of falling rock will be small enough to be contained by a concrete barrier.

The impact energy from falling rock is the sum of kinetic (translational) energy and rotational energy before it collides with the barrier. These parameters can be calculated from the recorded videos and also computer simulations of rocks can be useful in estimating the translational and rotational motion. The tests on the concrete barriers are
usually performed up to the impact energy absorption capacity of the barriers and this limit are usually determined by mean of experiments (Peila, et al., 1998; Grassl, et al., 2002; Hearn, et al., 1995).

A thorough literature search and preliminary rockfall analysis revealed that concrete barriers may not be able to contain high energy rockfalls. A general consensus is that the energy absorbed by concrete barriers is likely to be about 50 to 70 kJ (kN-m) or 18 to 26 ton-ft. A relation between impact energy absorbing capability and construction costs of concrete barriers is shown in Figure 2.19. It is demonstrated that while concrete barriers are relatively low energy dissipation systems, they are relatively inexpensive.

![Figure 2.19 Energy absorbed compared to the type of protection system (Maegawa, et al., 2011)](image-url)
CHAPTER III
PHASE I IMPACT TESTING

3.1 Introduction

This chapter describes the procedures implemented by the research team in the full-scale impact tests, which were conducted in two phases. In the first phase, which was performed in the summer of 2012, impact tests were conducted on standard PCBs and modified CIP concrete barriers currently used by ODOT. All tests were conducted on the premises of Duer Construction Company, located at 1016 Morse Street in Akron, Ohio. An aerial view of the test site is shown in Figure 3.1.

![Aerial view of the test site]

Figure 3.1  Aerial view of the test site

Two parallel pavement pads were constructed at the test site, one made of asphalt and the other made of concrete, in order to verify if there is any difference in the energy
absorption capacity of CIP concrete barriers if they are cast against different pavement types. The dimensions of both pads were 70 feet in length, 5 feet in width, and 9 inches thick; the pads were spaced 6 feet apart. The plan dimensions for the pavement pads are shown in Figure 3.2, and a picture showing the installation of the PCB and CIP concrete barriers along the pavements is shown in Figure 3.3.

![Figure 3.2 Plan dimensions of the pavement pads](image)

### 3.2 Methodology

The first phase of testing involved full-scale impact testing performed on current ODOT standard PCB and modified CIP concrete barriers. The setup of the impact tests consisted of each test rock suspended from a track hoe by a steel chain, and raised to the desired height to collide in a pendulum motion with the test concrete barrier, which was installed along one of the pavement pads (Figure 3.4).

![Figure 3.4](image)

Current ODOT standard PCBs are provided with reinforcement details that are designed to satisfy vehicular crash test requirements of TL-3 as per NCHRP Report 350. Similarly, unreinforced CIP concrete barriers are designed to satisfy TL-3 requirements. In this test program, one of the primary objectives is to quantify the energy absorption capacity of the ODOT modified concrete barriers. Five units of PCB conforming to the requirements of the current ODOT standard drawings were installed in sequence for the
impact tests; two separate 60-foot-long, 42-inch-tall CIP concrete test barriers were constructed on the site.

Figure 3.3 Concrete barriers with pavements

Figure 3.4 First phase impact test setup for PCB
3.2.1 ODOT Standard PCBs

The ODOT standard PCBs were provided by local precast company according to ODOT standard specifications. The standard compressive strength of concrete for these barriers is 4,000 psi. Each PCB was 32 inches in height and 12 ft. in length. The longitudinal elevation and cross-section of a typical barrier are shown in Figure 3.5 (extracted from ODOT Standard Drawing # RM 4.2 for WWF option). The PCB shown in the figure is reinforced with Welded Wire Fabric (WWF) at a spacing of 6 inches on center.

![Figure 3.5](image)

Figure 3.5  ODOT standard: a) PCB elevation, b) typical cross-section

The test setup of PCBs, including the placement and a typical cross-section, is shown in Figure 3.6.
3.2.2 Modified ODOT CIP Concrete Barrier

CIP concrete test barriers were constructed on the test site similar to modified concrete barriers with 4,000 psi concrete. The length of each CIP concrete test barrier was 60 ft. and the height was 42 inches. One CIP barrier was constructed along the concrete pavement and a second CIP barrier was constructed along the asphalt pavement. The reinforcement details used for the test barriers are shown in Figure 3.7. The size of both vertical and horizontal reinforcing bars was #5 with a diameter of 0.625 inch and a 0.31 in\(^2\) cross-sectional area. The horizontal reinforcement was provided at a spacing of 10.5” on each face of the barrier with a splice length of 2’-5” at the lap section (at the mid-length where a contraction joint was provided). The vertical reinforcement was provided at a spacing of 2 feet along each face of the test barriers for most of the barrier’s length except near drainage window, at the ends, and at the splice location. A drainage window that was 5 feet long and 4 inches high was provided at the center (mid-length) of the barrier.
Figure 3.7  Reinforcement details of CIP barriers: a) Reinforcement details for the CIP concrete barrier, b) Section detail: D-D, c) Section detail: C-C

Figure 3.8 shows details of the placement of the CIP concrete barriers along the concrete pavement and along the asphalt pavement.
3.2.3 Test Rocks

The “Test Rocks” in the impact testing were variable and consisted of reinforced concrete rocks, steel balls, and natural rock (Table 3.1). Details of each rock type are provided below:

**Reinforced concrete rocks:** Several manufactured “Test Rocks” were made from high strength concrete reinforced with steel bars and fibers in the materials laboratory at the University of Akron. The rocks were prepared in cubic, spherical and round shapes with variable masses.
Steel balls: Two steel balls weighing 3,800 lb and 2,070 lb were supplied by the company that owned the track hoe and crane.

Natural rock: A natural rock weighing 1,500 lb was also used.

Table 3.1 shows a summary of the details of the Test Rocks used in the analysis for the impact tests. The shape of each rock mainly describes the rock surface at the level at which the impact occurred with the test barrier. In addition, Figure 3.9 provides photos of the “Test Rocks”.

Table 3.1 Summary of the Test Rocks used in the impact tests

<table>
<thead>
<tr>
<th>Test Rock #</th>
<th>Reinforced Concrete Rocks (RCR)</th>
<th>Steel Balls (SB)</th>
<th>Natural Rock (NR)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test Rock #</td>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Dimension (in)</td>
<td>16</td>
<td>16</td>
<td>18</td>
</tr>
<tr>
<td>Weight (lb)</td>
<td>150</td>
<td>190</td>
<td>260</td>
</tr>
<tr>
<td>Density (lb/ft³)</td>
<td>128</td>
<td>163</td>
<td>155</td>
</tr>
<tr>
<td>Shape</td>
<td>RND</td>
<td>RND</td>
<td>RND</td>
</tr>
</tbody>
</table>

RND = Round, SPH = Spherical, CUB = Cube

Figure 3.9 Test Rocks used for impact tests
3.2.4 Data Acquisition System

Data acquisition systems such as accelerometers, strain gages, high-speed cameras, theodolite, and tapes were used during the field impact tests to collect data from the field for further analysis. The parameters obtained from the field impact tests include impact velocities, impact energies, strains, accelerometers, cracking patterns and failure modes of the test barriers.

i) Accelerometers

Two types of accelerometers, uniaxial and triaxial, were utilized for acquisition of test data. The uniaxial accelerometers were fixed to the back face of the barrier being tested to capture accelerations of the barrier in the direction of the impact. Plates with dimensions of 3 inches x 4 inches with ¼ inch thickness were glued to the surface of the barrier at the desired locations. Each plate had a threaded hole at the center. A uniaxial accelerometer was bolted to the each plate (Figure 3.10). A triaxial accelerometer was mounted inside each of the reinforced concrete test rock to capture the accelerations of the “Test Rock” during the impact (Figure 3.10). The resultant acceleration of the test rock during the impact was determined using the triaxial accelerometer.

![Figure 3.10 Uniaxial accelerometer installed on barrier (left) and triaxial accelerometer installed in the impacting ball (right)](image)
ii) Strain Gages

Strain gages were used to measure the strains on the surface of the concrete test barriers and in the reinforcing steel. To measure strain in concrete, the strain gages were attached to the surface of the barriers with high-strength epoxy. The strains in the reinforcing bars were measured at critical sections both on the vertical and horizontal rebars. Figure 3.11 shows the application of strain gages on the surface of barriers.

![Figure 3.11 Strain gages on concrete surface and rebars](image)

iii) Theodolite and Measuring Tape

A theodolite mounted on a tripod and measuring tape were used to correctly position the Test Rock vertically and horizontally before releasing it to impact the barrier. Correct positioning of the Test Rock before release was needed in order to achieve the desired impact energy for each impact test.

iv) High-Speed Camera

VISION high-speed cameras (PhaseSpace, Inc., San Leondro, CA) were used in the impact tests to record videos for later analysis. The videos were used to capture the motion of impacting rocks, the movement of concrete barriers and the crack patterns of the barriers resulting from the impact of the test rock. Two high-speed cameras were implemented to monitor the barrier, while a third camera monitored the motion of the rocks during the
impact with the barrier. The VISION cameras feature a global shutter. They were able to obtain images at 4.2 megapixel resolution (2048×2048 pixel) at a rate of 50 to 100 frames per second (fps). Images for a selected region of interest can also be reduced to a resolution 64 × 64 pixels with adjustable x and y position at a rate of 1000 fps.

v) Server and Laptops

Laptop computers (provided by Trillion Technology Services) and a compact server (provided by PhaseSpace, Inc.) were used for storing, processing, and analyzing images. The server incorporated an Intel® Core i7-3770 3.4 GHz quad-core processor with an Intel® HD 4000 graphics driver, a Debian/Linux operating system, and 4 GB of memory. The server and laptops were operated from a trailer parked at the test site. High-speed cameras were connected to the server using Category 6 (Cat6) LAN network cables and connectors. Two LAN cables were connected to each camera: one to supply power and one for transmitting data between the server and the camera.

vi) Image Processing and Analysis:

PHASESPACE software (PhaseSpace, Inc.) was used to operate multiple high-speed cameras at different resolutions and frame rates, record video footage filmed from the VISION high-speed cameras, and process the VISION camera data collected during the impact tests. This application allows individual storage of image frames as brief as 20 microseconds, and these frames can be saved in TIFF format for later analysis. ARAMIS non-contact optical 3D deformation measuring system (developed by Gesellschaft für Optische Messtechnik mbH, Braunschweig, Germany) was used to analyze the images obtained during the impact tests and to calculate and document the deformations and strains on the back face of the test barriers. ARAMIS recognizes the surface structure of the
measuring object from the digital camera images and assigns location coordinates to the image pixels, using a pre-impact image of the surface to represent the undeformed state of the object and comparing the subsequent digital images to calculate the displacement and deformation of the barrier for each frame captured during the tests.

3.2.5 Impact Test Procedure

Impact tests were performed on each concrete test barrier using the principle of pendulum motion shown in Figure 3.12. The Test Rock for each impact was raised to the desired height necessary for the calculated impact energy and was released. This allowed for free fall of the Test Rock in the form of a pendulum swing until impacting the barrier test section. The following general procedure was followed during the impact testing.

The desired impact energy to be delivered to the test barrier was increased gradually for a given ball size during the test to observe the progressive failure mechanisms of the barrier in Phase 1 of impact tests. Different Test Rock sizes were used in different stages of impact tests starting from the smallest rock sizes and lowest drop heights.

The first step is the calculation of the potential energy for impact tests. The potential energy, $E_{pot}$, of the Test Rock is given by:

$$E_{pot} = mgh$$  \hspace{1cm} 3.1$$

Where $m$ is the mass of the Test Rock, $g$ is the gravitational accelerations and $h$ is the drop height.

The second step is the calculation of the Kinetic energy. The kinetic energy ($E_{kin}$) of the Test Rock just before an impact is given by:

$$E_{kin} = \frac{1}{2}mv^2$$  \hspace{1cm} 3.2$$
Where \( m \) is the mass of the Test Rock, and \( v \) is the velocity of the swinging rock which is equal to the horizontal velocity at the bottommost position of the Test Rock where the impact occurred.

The impact velocity (\( v_{\text{impact}} \)) of the Test Rock can be calculated by considering the principle of conservation of energy, considering negligible energy loss during the falling of the rock.

\[
E = mgh = \frac{1}{2}mv_{\text{impact}}^2
\]

Which is equivalent to:

\[
v_{\text{impact}} = \sqrt{2gh}
\]

Where \( g \) is gravitational acceleration and \( h \) is the drop height.

Figure 3.12  Impact test

The impact force (\( F_{\text{impact}} \)) is calculated by considering the incremental velocity (\( \Delta v \)) and the incremental time (\( \Delta t \)) captured in the high-speed camera system. The rebound velocity of the Test Rock was considered negligible, making it consistent with the assumption that no energy is lost in the system during the impact.
The impact force is given by the following equation

$$F_{\text{impact}} = \frac{m\Delta v}{\Delta t} = -\frac{mv_i}{\Delta t}$$

Where $m$ is the mass of the ball, $v$ is the terminal impact velocity of the Test Rock assuming that the final velocity is zero (Test Rock comes to rest after impact). For this testing, the contact time can be estimated from the timestamp on the high-speed camera recordings and also can be calculated from the graph of the accelerometer record, which has typically a rising time to peak value and then a fall in velocity to reach a zero acceleration.

3.2.6 Field Impact Test Parameters

Impact testing was conducted with the same Test Rocks, data acquisition system, and test procedure for both the ODOT standard PCB and modified CIP concrete barriers. Impact tests were performed based on parameters that would represent actual rockfalls impacting concrete barriers. Parameters evaluated during the impact testing were:

- Rock size, shape and type
- Level of impact and location of impact (along longitudinal axis)
- Drop height, impact energy and impact velocity
- Number of impacts

3.2.6.1 Standard ODOT PCB Barrier

The testing of the standard ODOT PCB followed the general procedures outlined in Section 3.2.1 and consisted of five PCB units connected together as shown in Figure 3.13. The first set of impacts was at the center of the third PCB unit, the second set of impacts was at the connection between the second and third PCB units, and another set of impacts was at the center of the second PCB unit.
The Test Rock was made to strike the test PCB at different sections and levels by positioning the track hoe as needed. At any section, the impacts were made at two levels (called “top” and “bottom”), 6 inches and 24 inches, measured from the top of the barrier as shown in Figure 3.14. These impact levels were intended to correspond with the approximate points of contact at the mid-height of the Test Rock.

However, due to the lack of good control on the vertical wobble of the track hoe boom and the geometry of the slope of the bottom portion of the test PCBs, the impact occurred slightly off the target, but within about 3 to 4 inches of the targeted location. However, the impact locations were useful in investigating the strength of the barrier at different heights, as rockfall can strike a barrier at any section or level.

Figure 3.13  Impact locations along the PCB

Figure 3.14  Impact levels as marked along the barrier heights
3.2.6.2 Modified ODOT CIP Concrete Barriers

Similar to the impact locations described above for PCBs, the Test Rock was made to strike the CIP concrete barrier at different sections along the length of the barrier and different levels along the height at any particular section. Three initial impact locations along the longitudinal axis of each test barrier were made as shown in Figure 3.15. The first impact location was at the center (at 30 ft from the ends), the second impact location was at the contraction joint (at 20 ft. from the ends) and third impact location was at a longitudinal distance of 10 ft. from the edges of the test barriers. However, impacts were later made all along the 60 ft. length of each test barrier, including locations close to the end of the barrier to maximize the collection of test data.

The impact at a particular location did not seem to influence the energy absorption capacity at the adjacent sections if the impacts were spaced approximately 6 feet from each other, as long as the failure lines did not overlap. The impacts were performed at two vertical levels, 9 inches and 28 inches, as measured from top of the barrier up to the mid-point of each Test Rock (Figure 3.16). The pavement type was an additional data variable included during the CIP concrete barrier testing.

![Figure 3.15 Longitudinal impact locations of the CIP against the concrete pad](image-url)
3.2.6.3 Determination of Test Rock Sizes and Drop Heights

A combination of Test Rock mass and drop height determines the impact energy for any particular impact test. Based on the literature review, the minimum and maximum expected limits of the energy absorption capacities of typical PCBs and CIP concrete barriers were approximately 20 kJ and 70 kJ. Therefore, the same range of energies was expected for ODOT standard PCBs and modified ODOT CIP concrete barriers in Phase 1 of this investigation. The possible range of impact velocities available for different drop heights when the deployed track hoe was used are presented in Figures 3.17.
The energies that can be generated using the different Test Rocks are shown in Figure 3.18. The Figure also shows the energy absorption limit of 50kJ and 70kJ for concrete barriers that are obtained from a thorough literature review.

![Figure 3.18](image)

**Figure 3.18** Impact energy versus drop height

### 3.3 Impact Test Results

This section describes the field impact test results on the PCB and CIP concrete barriers. This includes impact velocity, impact energy, number of impacts and other field data analysis such as strain gages, high speed video camera.

#### 3.3.1 Results from the Impact Tests of Standard PCBs

ODOT practice is to place PCBs on the shoulder next to the edge of the pavement. Therefore, the lateral support for PCB barriers is only mobilized by friction between the bottom of the PCBs and the shoulder surface. The stability of PCBs against lateral loading
is provided mainly by its own self-weight, which would provide the stabilizing moment of resistance during tipping of the barrier system, friction at the base, and the rotation of linked barriers at the hinged joints between adjacent units. Under a rockfall impact from the ditch side, PCBs can slide toward the adjacent roadway, particularly when the impact occurs close to the joints.

This section outlines the results obtained from the impact tests performed on PCBs. A total of 45 impacts were made during Phase 1 tests. The concrete barrier was found to absorb the energy delivered by the Test Rocks up to certain number of progressively increasing impacts. Small Test Rocks from small drop heights were used initially, and the Test Rock sizes were gradually increased in later impacts until the concrete barrier failed. Table 3.2 summarizes the velocities and energies obtained from impact tests performed on PCBs by using various Test Rock sizes and impact locations.

Table 3.2  Drop heights, velocities and impact energies from impact tests on PCBs

<table>
<thead>
<tr>
<th>Hit #</th>
<th>Drop Height (ft)</th>
<th>Impact Velocity (ft/s)</th>
<th>Impact Energy (kJ)</th>
<th>Rock Mass (lb)</th>
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Table 3.2 Drop heights, velocities and impact energies from impact tests on PCBs (continued)

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<th>Rock Mass (lb)</th>
<th>Impact location</th>
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Figure 3.19 shows drop heights versus impact velocities of the Test Rock used in the tests on the PCBs. Figure 3.20 shows the drop height versus energy of the Test Rock.
used to test the PCBs. As the drop height increases, energy delivered by the Test Rock increases. The impact energy increases with the increase of the mass of the test rock and the drop height.

![Figure 3.19](image1.png)  
**Figure 3.19**  Drop heights versus impact velocities for PCB test specimens

![Figure 3.20](image2.png)  
**Figure 3.20**  Drop height versus impact energy for standard PCBs

It was observed from the impact tests that multiple impacts starting from the low energy impact to higher energy impact at the same location causes a significant amount of cracking and a weakening of the barrier thereby leading to reduced energy absorption. Therefore, it was decided in the subsequent tests to make a single impact at any particular
test location. This approach was expected to define the true energy absorption capacity of these barriers and the failure mode under single impact. In this context, a test barrier was considered to have failed when the barrier was mostly damaged/destroyed with extensive cracking and spalling of concrete while still being able to retain the Test Rock that caused the failure.

One impact test was conducted on each of the two PCBs using a 1270-lb Test Rock with a drop height of about 14 ft. Each barrier failed with a single impact at the center of the barrier. The maximum energy absorbed by the PCB was 24.7 kJ as shown in Table 3.3.

Table 3.3 Maximum energy absorption of the standard PCBs

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<tr>
<th>Hit #</th>
<th>Barrier</th>
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<th>Impact Energy (kJ)</th>
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3.3.2 Tri-axial Accelerometer of a Typical Test Rock on PCB

Figure 3.21 shows a typical accelerometer result recorded from a 260-lb Test Rock during an impact from drop height of 4 ft on PCBs at the center-top section. The peak accelerometer data was observed to be 500g and the impact contact duration was 0.002 sec.

![Figure 3.21 Typical accelerations of a Test Rock recorded during the testing of a PCB](image-url)
3.3.3 CIP Concrete Barrier Installed against Concrete Pavement

The impact tests on CIP concrete barrier installed against concrete pavement were conducted in August 2012. A total of 52 impacts were made during these tests. The impact energies in the tests were calculated based on the Test Rock weight and the corresponding drop height. The initial impacts were made using a Test Rock with a smaller weight that was released from different heights to strike the concrete barrier at the same location. The impact energy was increased progressively to study the performance of the barrier under multiple impacts. The impact energies calculated from the Test Rock and the corresponding drop heights during the tests of CIP concrete barriers are shown in Table 3.4.

Table 3.4  Results for CIP concrete barriers installed against concrete pavement

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<th>Rock Mass (lb)</th>
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Table 3.4  Results for CIP concrete barriers installed against concrete pavement
(continued)

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<td>3800</td>
<td>2'6&quot; from Right</td>
</tr>
<tr>
<td>51</td>
<td>8.6</td>
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<td>44.2</td>
<td>3800</td>
<td>2'6&quot; from Right</td>
</tr>
<tr>
<td>52</td>
<td>8.7</td>
<td>23.6</td>
<td>44.6</td>
<td>3800</td>
<td>2'6&quot; from Right</td>
</tr>
</tbody>
</table>
Figure 3.22 shows the drop height versus velocity for the impact tests conducted on the CIP concrete test barrier installed against concrete pavement (for the data included in Table 3.4). Similarly, Figure 3.23 shows the drop height versus impact energy for the same set of tests.

Figure 3.22  Impact velocity versus drop height for Test Rocks for CIP concrete barriers installed against concrete pavement

Figure 3.23  Impact energy versus drop height for the CIP concrete barrier installed against concrete pavement
3.3.4 CIP Installed against Asphalt Pavement

The 39 impact tests were conducted in August 2012 on the CIP concrete barrier cast against asphalt pavement. The energies in these impact tests were calculated from the Test Rock sizes and the corresponding drop heights. The initial impacts were made by using the smaller Test Rocks and releasing those from different drop heights. The impact energy was then increased progressively to study the performance of the barrier under multiple impacts. The impact energies calculated in that manner for each impact are listed in the Table 3.5.

Table 3.5   Drop height, velocity, and energy for the impact tests on CIP barrier installed against the asphalt pavement

<table>
<thead>
<tr>
<th>Hit #</th>
<th>Drop Height (ft)</th>
<th>Impact Velocity (ft/s)</th>
<th>Rock Mass (lbs)</th>
<th>Impact Energy (KJ)</th>
<th>Impact location</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4.1</td>
<td>16.3</td>
<td>150</td>
<td>0.8</td>
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</tr>
<tr>
<td>2</td>
<td>4.1</td>
<td>16.3</td>
<td>150</td>
<td>0.8</td>
<td>1 – Top</td>
</tr>
<tr>
<td>3</td>
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<td>150</td>
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<tr>
<td>4</td>
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<td>150</td>
<td>1.3</td>
<td>1 – Top</td>
</tr>
<tr>
<td>5</td>
<td>6.4</td>
<td>20.3</td>
<td>150</td>
<td>1.3</td>
<td>1 – Bottom</td>
</tr>
<tr>
<td>6</td>
<td>3.6</td>
<td>15.3</td>
<td>190</td>
<td>0.9</td>
<td>1 – Top</td>
</tr>
<tr>
<td>7</td>
<td>3.6</td>
<td>15.2</td>
<td>190</td>
<td>0.9</td>
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</tr>
<tr>
<td>8</td>
<td>5.9</td>
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<td>190</td>
<td>1.5</td>
<td>1 – Top</td>
</tr>
<tr>
<td>9</td>
<td>5.5</td>
<td>18.9</td>
<td>190</td>
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<td>1 – Bottom</td>
</tr>
<tr>
<td>10</td>
<td>2.4</td>
<td>12.4</td>
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<td>1 – Top</td>
</tr>
<tr>
<td>11</td>
<td>3.1</td>
<td>14.0</td>
<td>260</td>
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<td>1 – Bottom</td>
</tr>
<tr>
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<td>6.2</td>
<td>20.0</td>
<td>260</td>
<td>2.2</td>
<td>1 – Top</td>
</tr>
<tr>
<td>13</td>
<td>5.9</td>
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<td>3.6</td>
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</tr>
<tr>
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<tr>
<td>17</td>
<td>5.1</td>
<td>18.0</td>
<td>312</td>
<td>2.1</td>
<td>1 – Bottom</td>
</tr>
<tr>
<td>18</td>
<td>5.8</td>
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<td>24.8</td>
<td>312</td>
<td>4.1</td>
<td>1 – Top</td>
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<td>20</td>
<td>11.2</td>
<td>26.8</td>
<td>312</td>
<td>4.7</td>
<td>1 – Bottom</td>
</tr>
</tbody>
</table>
Table 3.5  Drop height, velocity, and energy for the impact tests on CIP barrier installed against the asphalt pavement (continued)

<table>
<thead>
<tr>
<th>Hit #</th>
<th>Drop Height (ft)</th>
<th>Impact Velocity (ft/s)</th>
<th>Rock Mass (lbs)</th>
<th>Impact Energy (KJ)</th>
<th>Impact location</th>
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<td>720</td>
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<tr>
<td>23</td>
<td>5.9</td>
<td>19.6</td>
<td>1270</td>
<td>10.2</td>
<td>1 – Center</td>
</tr>
<tr>
<td>24</td>
<td>10.5</td>
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<td>1270</td>
<td>18.1</td>
<td>1 – Center</td>
</tr>
<tr>
<td>25</td>
<td>13.6</td>
<td>29.5</td>
<td>1270</td>
<td>23.3</td>
<td>1 – Center</td>
</tr>
<tr>
<td>26</td>
<td>13.8</td>
<td>29.8</td>
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<tr>
<td>27</td>
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<tr>
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<td>2 – Center</td>
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<td>29</td>
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<tr>
<td>31</td>
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<tr>
<td>32</td>
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<tr>
<td>33</td>
<td>12.7</td>
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<td>37.9</td>
<td>1 – Center</td>
</tr>
<tr>
<td>34</td>
<td>13.7</td>
<td>29.8</td>
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<td>41.0</td>
<td>1 – Center</td>
</tr>
<tr>
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<td>12.9</td>
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<td>38.5</td>
<td>1 – Center</td>
</tr>
<tr>
<td>36</td>
<td>11.9</td>
<td>27.5</td>
<td>2070</td>
<td>32.9</td>
<td>12' from Right</td>
</tr>
<tr>
<td>37</td>
<td>11.6</td>
<td>27.5</td>
<td>2070</td>
<td>32.9</td>
<td>12' from Right</td>
</tr>
<tr>
<td>38</td>
<td>11.4</td>
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<td>2070</td>
<td>27.2</td>
<td>36&quot; from Right</td>
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<tr>
<td>39</td>
<td>12.2</td>
<td>29.2</td>
<td>2070</td>
<td>37.1</td>
<td>36&quot; from Right</td>
</tr>
</tbody>
</table>

Plots showing the impact velocity vs. drop height and impact energy vs. drop heights are presented in Figures 3.24 and 3.25 respectively.

![Figure 3.24](image-url)  
Figure 3.24  Impact velocity versus drop height for CIP concrete barriers installed along asphalt pavement
3.3.4.1 Results from strain gages

Strains were obtained from the strain gages attached to the steel reinforcing bars embedded in the CIP test barriers and also on the surface of the barriers. Figure 3.26 shows a typical output of strain gages when a 720-lb Test Rock was dropped from a drop height of 14.3-ft and impacted at the mid-length of the barrier. The peak strain for longitudinal rebar for this impact was 530 µε and the peak concrete strain on the surface of barrier is 480 µε. The corresponding impact energy from Table 3.5 is 13.9 kJ.

This figure indicates that strains in steel reinforcing bars and concrete are reasonably small compared to the corresponding yield strain of steel (2,000 µε or 0.002) and ultimate flexural compressive strain of concrete (3,000 µε or 0.003), respectively.
3.4 Discussion of the Impact Tests on ODOT Standard PCBs and Modified CIPs

Repeated low energy impacts at the same location cause significant cracking and weakening of concrete barriers, leading to a reduction in energy absorption capacity of concrete barriers compared to their energy absorption capacity due to a high-energy single impact event. This means that concrete barriers can withstand much larger impact energy if no prior impact occurred at the same location. The practical implication of this finding is that it is preferable to replace PCBs after a rockfall has impacted the barrier even if the barrier was hit by a low-energy rockfall that caused some visible cracking and minor damage. Similarly, it is also preferred to repair or replace the portion of a CIP barrier which was subjected to a low-energy impact, even if there are no visible signs of severe damage or failure.

3.4.1 Failure Modes of PCB

Figure 3.27 presents the failure modes for PCBs. The PCB test specimens failed by extensive cracking followed by crumbling of concrete (Figure 3.27a and c), rupture of steel
welded wire fabric (WWF) as seen in Figure 3.27c, and excessive sliding of the base (Figure 3.27b). Concrete crumbling was particularly severe in the unreinforced areas of the PCB test specimen (Figure 3.27c).

The impacts at the hinges did not impede the energy absorption any less than impacts elsewhere in the test sections. However, spalling of concrete was observed at the ends of #4 bars (Figure 3.27d). Therefore, suitable adjustments to the reinforcing details are needed to improve the performance of PCBs under impact loading.

Figure 3.27  Failure modes of PCB

It was observed in the impact tests that multiple impacts at the same location of the test section lead to the failure of concrete barriers under a significantly smaller energy impact compared to a single impact with a larger energy.
3.4.2 Failure Modes of CIP Concrete Barriers

CIP concrete barrier specimens also failed by extensive cracking followed by spalling of concrete (Figure 3.28). The shape of the failed segment of the wall indicated that the failure was caused by punching shear or by the development of yield lines. The angle of failure lines with respect to the horizontal was about 30 to 40 degrees, as seen in the figure. The maximum impact energy that was delivered to the CIP concrete test barriers was recorded to be 57.5 kJ (42.6 kip-ft) when a single impact was made at a section. A severe lack of bond between the embedded epoxy coated bars and the surrounding concrete was noted as can be seen in Figure 3.28. Crumbling of concrete was also observed at the locations where the reinforcing bars were provided at large spacing (18 to 24 inches).

![Figure 3.28 Failure modes for CIP barrier](image)

Impacts at a spacing of 6 to 8 feet along the length of the barrier (Figure 3.29) were resisted well by the CIP barriers irrespective of the damage at the adjacent sections outside of the 6 to 8 feet distance from the center of the impact, demonstrating that closely spaced impacts can be resisted by CIP barriers satisfactorily even if failure occurred at the adjacent locations. The length of influence of each impact is defined by the failure plane, which was about 6 to 8 ft. long at the top along the length of the barrier.
3.4.3 Performance of ODOT Standard PCBs under Impact

The two PCB test barriers that were each subjected to a single impact were able to withstand impact energies of 23.5 kJ and 24.7 kJ respectively. The impact energy delivered to the second test barrier (PCB-2) was based on the impact test on the first test barrier (PCB-1). The increased impact energy delivered to PCB-2 was successfully withstood by PCB-2. Therefore the maximum impact energy absorbed by PCB test specimens is considered to be 24.7 kJ (18.22 kip-ft.). PCB test specimens were able to contain the Test Rock and did not allow it to jump over the barrier as long as the impact occurred near or below the $\frac{2}{3}$rd of the height of the barrier from the PCB base. The PCB test specimens tipped about their toes, losing contact at their heels. PCBs rotated significantly, causing a lateral displacement of up to 10 inches at the top. However, they did not overturn due to the linking of adjacent PCB units through hinged joints, which engaged multiple units in the resisting the impact. Sliding by as much as 12 to 15 inches was observed, suggesting that the placement of PCBs at the edge of the pavement may result in sliding of the PCBs.
closer to the traffic when a large impact rockfall occurs. Such sliding of PCBs may be acceptable as long as it is planned to keep the PCB placement 18 inches from the traffic lane. The energy delivered by the Test Rock was also dissipated through kinking (rotation about vertical axis) at the hinged joints, suggesting that shorter length PCBs may be more effective in dissipating impact energy than longer PCBs.

WWF provided in accordance with the details shown in ODOT standard PCB drawings ruptured under impact loading. Severe spalling and crumbling of concrete occurred at the unreinforced regions within the PCBs. Disintegration of concrete by spalling and crumbling seemed to be the primary cause for the PCBs to become ineffective in absorbing impact energies and for the loss of structural integrity. This observed behavior leads to the suggestion that current reinforcement details may need to be modified to reduce the unreinforced areas within PCBs so as to improve the energy absorption capacity and also to reduce cracking and spalling of concrete.

The bolted hinge details of the ODOT standard PCB seemed to work well. The response of the test specimens under a direct impact at the hinge was found to be similar to the response of the specimen when the impact was at mid-length of the middle PCB section. However, spalling was observed at the ends of #4 bars near the hinge. Kinking at the hinges at the ends of the PCBs was a dominant mode of deformation when the impact was made at a hinge.

3.4.4 Performance of ODOT Standard Modified CIP barriers

The CIP test barriers of 42” height were able to resist impact energy of up to 57.5 kJ (42.6 kip-ft) if the impact is based on single occurrence at the location. The CIP test barrier was able to contain the Test Rock and did not allow it to jump over the barrier as
long as the impact occurred near or below the \( \frac{2}{3} \)rd of the height of the barrier from the top of the barrier footing. The CIP test barriers appeared to have slightly tipped about the toe, losing contact briefly at the heel and rotating by a small amount particularly when the impact was close to the ends (within 5 to 10 ft.) of the barrier. However, the test barriers returned to the stable undeformed position after the impulse died down, and there was no overturning or any permanent lateral deformation.

The vertical face of the adjacent pavement pad (concrete or asphalt) against which the footing of the test barrier was cast was able to provide the required horizontal reaction to the footing at the base of the barrier through bearing on the vertical joint. For the most part, the unreinforced footing of the CIP barrier functioned well without exhibiting any cracking or distress. There was no indication of failure at the cold joint between the footing and the stem of the barrier, indicating that the shear transfer at the cold joint was adequate.

From these tests, it was concluded that the current detailing practice of ODOT may need to be revisited. Epoxy-coated reinforcing bars provided in the test barriers demonstrated a severe lack of bond with the surrounding concrete under impact loading. Extensive cracking, spalling and crumbling of concrete seemed to be one of the primary causes of the barrier failures. Cracking and spalling mostly occurred in the unreinforced areas of the barrier. Therefore, placing reinforcing bars at closer spacing is expected to reduce the tendency of the concrete in the barriers to crack and spall. Also, the vertical and horizontal reinforcement is generally not able to reach yield strains, meaning that #5 bars are not fully utilized under impact loads. This is possibly because of (i) cracking and spalling of concrete, which causes the concrete to lose its confining capacity to provide the required bond strength needed for the reinforcing bars to develop the required internal
tensile force, and (ii) the inferior bond strength of epoxy-coated bars particularly under impact loading. Both these two perceived problems may be minimized by reducing the diameter of the reinforcing bars and the corresponding spacing in order to eventually improvement in the energy absorption capacity. Reduced diameter with reduced spacing is expected to lead to improved impact energy absorption capacity of concrete barriers. Reinforcing bars of #4 at a spacing of 15 inches on center would be a suitable alternative to #5 bars at a spacing of 24 inches.

Some ODOT districts use hairpin (inverted U-shaped) reinforcing bars at the top of CIP barriers, but this is not an ODOT standard requirement. The test barriers in this study were not provided with hairpins at the top during the 2012 tests. Consequently, the failure mode of the test barriers revealed that the top of the test barrier can sometimes split in the longitudinal direction under the impact loading, with cracks developing along the length of the barrier at the top. A Hairpin type of reinforcement at the top is expected to reduce this tendency of concrete to split longitudinally.

3.4.5 Effect of Pavement Type

Both concrete pavement and asphalt pavement were able to provide adequate lateral support to the footing of the CIP barriers. The nine-inch-thick concrete pavement was made from plain concrete and showed many transverse cracks due the impacts, but it remained intact during the entire testing program; this finding suggests that the barriers were able to develop the required lateral reaction at the footing level. The asphalt pavement appeared to be more impact resistant, but it was difficult to quantify the difference in performance due to the pavement type, if any. No cracks were observed in the asphalt pavement.
3.4.6 Influence of Test Rock Type

Essentially two types Test Rocks (cubic or spherical) are available for evaluation of result of impact interactions with the concrete barrier. Cubical type Test Rocks consists of six faces with octagonal shaped surfaces and eight triangular shaped surfaces. These 14 surfaces create 36 straight edges and the spherical type Test Rocks consist of mostly round surfaces. When the cubical type Test Rock were utilized, typically either the flat edge or straight edge struck the barrier. For flat face impacts, the contact area is larger, resulting in lower impact forces on the barriers and less damage. If one of the straight edges impacted, the sharp edge results in local damage to the barrier compared to flat face impact. When spherical type Test Rocks were utilized, they caused more effective delivery of impact energy and maximum damage to the test barriers compared to the cubic type Test Rocks.

This suggests that the impact tests conducted with spherical shaped Test Rock (or a rock with a spherical impacting surface) is more severe than a rock with multiple edges or flat surfaces. Spherical rocks underestimate the impact performance of test barriers, meaning that the energy absorption limit of a concrete barrier is a lower bound limit when determined with a spherical shaped test rock. This is because the spherical rocks deliver higher impact energy than their counterparts with straight edges and/or flat surfaces. If the rocks in a rockfall from a natural slope are not spherical in shape, the energies delivered to the concrete barrier are likely to be somewhat smaller than those delivered by round (spherical) rocks.

In this investigation, the natural rock shattered into pieces upon impact. The sharp edge of the natural rock also provided a larger length of contact at the time of impact, making the impact less severe. The impact energy delivered by the natural rock was found
to be smaller than that of the corresponding concrete Test Rock of the same mass. The manufactured Test Rocks are a better representation of the moremassively bedded sandstone and limestone rocks which will not fragment upon impact. These types of materials are considered the critical acceptance for the lower bound thresholds for the design criteria (i.e. worst case).

3.4.7 Location and Number of Impacts

The vertical location of the impact influenced the amount of energy absorbed by the barrier for both PCBs and CIP concrete barriers. An impact closer to the top of the barrier developed a smaller length of failure plane; therefore, failure occurred at a smaller impact energy. The energy absorbed was much larger when the impact occurred at about the mid-height of the barrier.

The horizontal location of the impact along the barrier length did not influence the energy absorption to any notable degree. Adjacent impacts at a spacing of about 6 to 8 ft. did not reduce the impact resistance of the barrier in any way because the failure planes did not overlap. The energy absorption was diminished when the impact was delivered closer to the ends of the barrier, suggesting that it is better to disregard the energy absorption capacity in the 6 foot length at the end of the barrier. Alternatively, the barrier may be extended by at least 6 feet beyond where it is theoretically needed. In this case, the failure mode changed from punching to splitting within the last 6 feet of the concrete barrier.

Repeated impacts at one location caused cracking and softening of the barrier, leading to failure at impact energies much smaller than those corresponding to the location that is subjected to a single impact.
3.4.8 Drop Heights and Impact Energies

It was the higher energy impacts that caused failure of the barriers. The velocity (which depends on the drop height alone) did not affect the failure mode or the failure energy. In other words, a high-velocity impact and a low-velocity impact had similar effects on the concrete barriers as long as the energy delivered to the barrier was the same.

3.5 Phase 1 Conclusions and Recommendations

The results from Phase 1 impact testing showed extensive cracking and spalling of concrete during impacts. This section presented the important conclusions from the Phase 1 impact testing and recommendations which included the need for Phase 2 impact testing on revised designs of concrete barriers.

3.5.1 Phase 1 Conclusions

The following were the most important findings from Phase 1 impact testing for PCBs and CIP concrete barriers.

1) ODOT Standard PCBs

- PBCs made according to current ODOT standard details and specifications are suitable for maximum impact energy of up to 24 kJ (17.9 kip-ft).
- It is feasible to make improvements to the energy absorption capacity of PCBs by making suitable changes to the detailing of PCBs.

2) Modified ODOT CIP Barriers

- CIP barriers of 42” height made according to the modified ODOT traffic barrier details and specifications are able to resist impact energy up to 55 kJ (41 kip-ft).
- Detailing of CIP barriers may be modified to improve the performance under impact loading.
3.5.2 Need for Phase 2 Impact Tests

Severe spalling and crumbling of concrete was observed for PCBs and modified CIP test barriers under impact loading in Phase 1 tests. The spacing of the reinforcing bars for CIP barriers was found to be excessive. These test results suggest that the energy absorption capacity of the barriers may be increased by reducing the diameter of the reinforcing bars and reducing the spacing both vertically and horizontally.

The severe lack of bond between epoxy coated bars and the surrounding concrete was perceived as a serious problem under rockfall impact. The use of polypropylene or steel fiber in concrete was expected to improve the energy absorption capacity of PCBs and CIP barriers. However, these expectations were needed to be verified through additional impact testing in Phase 2.
CHAPTER IV
PHASE 2 IMPACT TESTING

4.1 Introduction

The Phase 1 tests identified some deficiencies in PCB and CIP concrete barriers that affected their ability to resist impact loading. A test plan was developed for Phase 2 to address the severe cracking and spalling of concrete and the lack of adequate bond when epoxy-coated bars were used. The Phase 2 test plan was designed with the intention of improving the energy absorption capacity of concrete barriers by adopting suitable modifications to the current designs.

In Phase 2 of the testing program, impact tests were conducted using concrete barriers with alternative design in summer of 2013. The testing in Phase 2 utilized larger drop heights resulting in greater impact energies to evaluate if the alternative designs could withstand higher energy impacts. While this testing was conducted with similar testing procedures as those used in Phase 1 during the summer of 2012, the scale of the testing had to be increased accordingly.

The following variables were included in the test program:

- Concrete material types
- Reinforcement details (bar spacing and bar size)
- Smooth/deformed WWF (welded wire fabric)
- Use of reinforcing bars vs. use of WWF
• Effects of epoxy coating on reinforcements
• Impact energies and impact velocities
• Impact locations

4.2 Methodology

Similar impact test setup was followed except that the higher drop height for higher impact energies.

4.2.1 PCBs

The PCBs tested in Phase 2 conform to ODOT’s standard outer dimensions of a 32-inch-tall traffic barrier (ODOT Drawing No. RM4.2). These barriers are currently made with 4,000 psi plain concrete and internally reinforced with steel reinforcing bars or WWF option. The material and conceptual details were modified without changing the outer geometry to evaluate designs with expected greater maximum impact energy absorption due to the proposed modifications. Two PCBs were evaluated for each alternative design with the following different options:

a) ODOT standard barrier (mesh option)
b) ODOT standard barrier (mesh option) plus polypropylene fibers
c) ODOT standard barrier (mesh option) plus steel fibers
d) ODOT standard barrier (mesh option) with foam board (see Figure 4.1)
e) ODOT standard barrier (mesh option) with foam board and with steel fiber

A summary of the PCBs included in Phase 2 testing is provided in Table 4.1. The first column listed the serial number of the PCBs. The third column shows the design options. Column 4 shows the fibers types and column 5 shows the dosage of fibers in concrete mix in lb/yd³. The last column shows the design comments.
Figure 4.1 PCB with foam board, elevation (left) and cross-section (right)

Table 4.1 Summary of the PCBs for impact tests

<table>
<thead>
<tr>
<th>PCB</th>
<th>Option</th>
<th>Description</th>
<th>Fiber type</th>
<th>Dosage</th>
<th>Comment</th>
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<td>ODOT Standard RM</td>
</tr>
<tr>
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<td></td>
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<td>None</td>
<td>NA</td>
<td>ODOT Standard RM</td>
</tr>
<tr>
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<td>b</td>
<td>PCB + PP Fiber 1</td>
<td>Polypropylene</td>
<td>7.5 lb/yd</td>
<td>ODOT Standard RM</td>
</tr>
<tr>
<td>4</td>
<td></td>
<td>PCB + PP Fiber 2</td>
<td>Polypropylene</td>
<td>7.5 lb/yd</td>
<td>ODOT Standard RM</td>
</tr>
<tr>
<td>5</td>
<td>c</td>
<td>PCB + Steel</td>
<td>Steel Fiber</td>
<td>40 lb/yd$^3$</td>
<td>ODOT Standard RM</td>
</tr>
<tr>
<td>6</td>
<td></td>
<td>PCB + Steel</td>
<td>Steel Fiber</td>
<td>40 lb/yd$^3$</td>
<td>ODOT Standard RM</td>
</tr>
<tr>
<td>7</td>
<td>d</td>
<td>PCB + Foam</td>
<td>None</td>
<td>NA</td>
<td>With Foam Board</td>
</tr>
<tr>
<td>8</td>
<td></td>
<td>PCB + Foam</td>
<td>None</td>
<td>NA</td>
<td>With Foam Board</td>
</tr>
<tr>
<td>9</td>
<td>e</td>
<td>PCB + Foam</td>
<td>Steel Fiber</td>
<td>40 lb/yd$^3$</td>
<td>With Foam Board</td>
</tr>
<tr>
<td>10</td>
<td></td>
<td>PCB + Foam</td>
<td>Steel Fiber</td>
<td>40 lb/yd$^3$</td>
<td>With Foam Board</td>
</tr>
</tbody>
</table>

The impact test setup used in Phase 2 is shown in Figure 4.2 which is essentially the same general set up utilized in Phase 1, except a crane was necessary to achieve a higher drop height resulting in greater impact energies. It consists of a crane, the backhoe to raise the Test Rock to the required level and a Test Rock of a given weight which is hung from the crane using a steel chain. Figure 4.3 shows the layout of the PCBs in the test setup, with
the test unit installed at the center. Figure 4.4 shows the PCB as installed at the test site. Impact tests were conducted on the middle units, while the two units on each side of the middle units provided continuity and some restraint.

![Impact test setup for Phase 2 testing](image.png)

**Figure 4.2** Impact test setup for Phase 2 testing

![Details of setup for PCBs](image.png)

**Figure 4.3** Details of setup for PCBs
4.2.2 CIP Concrete Barrier

Ten alternative designs for CIP barriers were tested in five test unit configurations for Phase 2. These alternatives provided the same external geometry necessary to satisfy traffic standards, but evaluated different reinforcement and material options. The test barriers were 42 inches in height and 40 feet in length constructed such that the right and left sides had different construction alternatives (reinforcement, material, or both) to minimize construction efforts. The first four barrier designs were constructed from #4 rebar on the right section and WWF on the left section with a drainage window at the center of each section (half length). Figure 4.5 presents the general layout of the test barrier. The fifth barrier design used WWF on both sides; the WWF on the left hand side had epoxy
coating, and the WWF on the right side had no coating. A 4-foot overlap between the left side and right side reinforcement was constructed in the center of the test unit.

The wet concrete used to make the test barriers included (i) normal concrete with no fiber, (ii) concrete with 7.5 lbs/yd3 of polypropylene fiber, or (iii) concrete with 40 lbs/yd3 of steel fiber.

![Diagram of concrete barrier](image)

**Figure 4.5** Typical CIP concrete barrier

i) **CIP Concrete Barrier Reinforced with Bars and WWF**

The 40-foot test barriers were constructed with the right side (20-foot section) reinforced with #4 bars and the left side (20-foot section) reinforced with WWF. At the center of the concrete barrier, there is 4-foot overlap of rebar and WWF from either side. Figure 4.6 shows the reinforcing bar cage before concreting and concrete barriers. Two layers of plain 6”×12” WWF were used as reinforcements on the left half section of the barrier. Two layers of WWF were used on each side of barrier cross-section. One additional layer was used along the footing of the barrier.

ii) **CIP Concrete Barrier Reinforced with Fibers**

This concrete barrier was constructed using the epoxy-coated bars on the left half and epoxy-coated deformed WWF on the right half. Two types of fibers were used: steel fibers and polypropylene fibers.
iii) CIP Concrete Barrier Reinforced with Deformed WWF

This barrier was constructed similar to the barriers discussed above using deformed WWF as its reinforcement. The left half the barrier had epoxy coating on the WWF and the right half of the barrier had no epoxy coating on the WWF. The details of the CIP test units are presented in Table 4.2.

Table 4.2 Summary of the CIP concrete barriers tested in Phase 2

<table>
<thead>
<tr>
<th>Unit No.</th>
<th>Left half</th>
<th>Right half</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIP-1</td>
<td>WWF 6 × 6 W4.0 × W4.0 - 3 layers with epoxy coating</td>
<td>Revised bar details with #4 @ 12” o/c with epoxy coating</td>
<td>No fiber, but epoxy coated bars</td>
</tr>
<tr>
<td>CIP-2</td>
<td>WWF 6 × 12 W8.3 × W4.0 plus another layer of WWF 12 × 6 W4.0 × W8.3 both without epoxy coating</td>
<td>Revised bar details with #4 @ 12” o/c without epoxy coating</td>
<td>No fiber, and no epoxy coated bars</td>
</tr>
<tr>
<td>CIP-3</td>
<td>WWF 4 × 4 D7.0 × D7.0 with epoxy coating and with steel fiber at a dosage of 40 lb per yd³</td>
<td>Revised bar details with #4 @ 12” o/c with epoxy coating with steel fiber</td>
<td>With steel fiber, and with epoxy coating on bars</td>
</tr>
<tr>
<td>CIP-4</td>
<td>WWF 4 × 4 D7.0 × D7.0 with epoxy coating and with PP fiber at a dosage of 7.5 lb per yd³</td>
<td>Revised bar details with #4 @ 12” o/c with epoxy coating with PP fiber</td>
<td>With PP fiber, and with epoxy coating on bars</td>
</tr>
<tr>
<td>CIP-5</td>
<td>WWF 4 × 4 D7.0 × D7.0 with epoxy coating and without fiber</td>
<td>WWF 4 × 4 D7.0 × D7.0 with no epoxy coating or fiber</td>
<td>No fiber</td>
</tr>
</tbody>
</table>
Reinforcement detailing: The detail of reinforcing bars for the right section and the WWF for the left section are shown in Figure 4.7. As can be noticed in this figure, there is overlap in a 4-foot section at the center of the barrier.

Figure 4.7  Reinforcement detailing over the 40 feet length of the CIP barrier

Figures 4.8 and 4.9 gives the reinforcement detailing for the left half section of the CIP concrete barriers and the right half section of CIP concrete barriers respectively.
Figure 4.8  Reinforcement detailing for the left section of the CIP concrete barrier: a) Elevation: 4" × 4" D7 WWF; b) Section details
4.2.2.1 Test Setup for CIP Concrete Barrier

The test setup for the CIP concrete barrier is shown in Figure 4.10. The black dots painted on the barriers and Test Rocks were used to capture the dynamic strains and cracks during impact by ARAMIS image analysis software. Two PCB units were connected to the ends of the barrier to simulate the continuity of the barriers for the impact near the edge of the CIP concrete barrier.
4.2.3 Test Rock Sizes and Shapes

One round steel ball and another pear-shaped steel ball were used in the Phase 2 tests for the revised barriers. Black dots were painted on the surface of the balls to capture the motion of the balls just before and after an impact as shown in Figure 4.11. At the level of impact, the test rocks had a round shape to ensure that the maximum amount of impact would be delivered to the barriers.

The smaller ball, Test Rock #8, was used for the impact tests on PCBs. The larger ball, Test Rock #9, was used for the impact tests on CIP concrete barriers. From a typical drop height, Test Rock #9 deliver larger impact energies since its weight is larger than Test Rock #8.
4.2.4 Drop Height and Impact Energy

The drop heights were varied in order to deliver the required impact energy: up to 45 kJ on PCB with Test Rock #8 and 180 kJ for CIP concrete barrier with Test Rock #9. For a given height, Test Rock #9 delivers larger impact energy than Test Rock #8. The larger drop height available during Phase 2 impact tests is illustrated in Figure 4.12.
4.2.5 Data Acquisition and Image Processing

The instrumentation used in Phase 2 of the impact test program comprised of same instruments as in Phase 1 (Section 3.2.4). The triaxial accelerometer were not used in the Test Rock. Black dots were painted on the barriers and Test Rocks to capture the dynamic strains and cracks during impact by ARAMIS image analysis software.

4.2.6 Number of Impacts and Locations

For PCBs, a single impact was made at the mid-length of the center unit of the test barrier system as shown in Figure 4.13 based on the estimated impact energy. Based on the results from the first impact, the impact was repeated on the identical barrier to duplicate the test.

![Figure 4.13 Impact location and level for the PCBs](image)

For CIP concrete barriers, impacts were made at different locations on the barriers. The impact tests were started from the right end of the barrier and continued to the left end of the barrier.

Many key impact locations were identified to test the CIP concrete barriers. Five impact locations were selected for the first two barriers, which were the barriers reinforced with reinforcing bars and plain WWF with/without epoxy coating. Figure 4.14 shows the five locations of the impacts along CIP-1 and CIP-2.

Three impact locations were selected to study CIP-3 and CIP-4. These concrete barriers had epoxy-coated reinforcing bars on the right section and deformed WWF on the
left section with polypropylene or steel fibers. The three locations of impact for CIP-3 and CIP-4 are shown in Figure 4.15.

![Figure 4.14 Five impact locations for CIP-1 and CIP-2](image)

![Figure 4.15 Impact locations for CIP-3 and CIP-4 at three locations](image)

Six impact locations were selected to study the performance of CIP-5 concrete barrier reinforced with deformed WWF. The second and seventh impacts were made at the same location as shown in Figure 4.16 to repeat a test at the same location.

![Figure 4.16 Impact locations for CIP-5 barrier at six locations](image)
4.3 Impact Test Results for Revised PCBs

The results obtained from the field impact tests performed on PCBs during Phase 2 impact testing are described in this section.

4.3.1 ODOT Standard PCB

Impact tests were performed on two PCBs reinforced with the WWF option as specified in the relevant ODOT standard drawings. These tests were conducted on PCBs without any modifications. A single impact was made on the first standard PCB by releasing the Test Rock from a drop height of 7'-8’. The impact energy absorbed by the barrier was 21.5 kJ. During the impact on the second standard PCB, the Test Rock was released from a drop height of 8'-0”. The impact energy delivered to the test barrier was 22.5 kJ.

While there was extensive cracking of PCB, splashing of concrete was not as severe under a single high energy impact as it was under multiple impacts recorded in the Phase 1 tests while the PCB specimens failed and broke into several pieces, demonstrating a total loss of structural integrity (Figure 4.17), they were able to prevent the Test Rock from spilling over to the back of the test barrier.

Figure 4.17  ODOT standard PCB: impact side (left), pavement side (right)
4.3.2 ODOT Standard PCBs with Polypropylene Fiber

Impact tests were performed on two ODOT standard design PCBs with WWF option that were constructed using concrete that incorporated polypropylene fibers. A single impact test was performed on each barrier. Figure 4.18 shows the typical failure mode of these PCBs. The energy absorption capacities of the barriers were similar, and similar failure patterns were observed from the two duplicate tests. During the first impact, the Test Rock was released from a height of 8’-7”. The resulting impact energy delivered to the barrier was 24.1 kJ. During the second impact test, the Test Rock was released from a height of 8’-9” and the resulting impact energy was 24.6 kJ.

![Figure 4.18 Typical PCB with polypropylene fibers: impact side (left) and pavement side (right)](image)

4.3.3 ODOT Standard PCB with Steel Fiber

Impact tests were performed on PCB test specimens with ODOT standard details with WWF option, and with the addition of steel fibers in the concrete. A single impact was made on each barrier. Typical failure mode of the test barriers with steel fiber is shown in Figure 4.19. During the first impact test, the Test Rock was released from a height of 11’-0” which resulted in impact energy of 30.9 kJ. During the second impact test, the Test Rock was released from a height of 13’-4” and the resulting impact energy was 37.4 kJ.
When compared to ODOT standard PCB without fiber, this is a nearly 70% improvement in energy absorption capacity.

Figure 4.19  Failure Mode of ODOT standard PCB with steel fibers: impact side (left) and pavement side (right)

4.3.4 PCBs with Foam Board

Impact tests were performed on PCBs constructed to conform to the outer geometry of an ODOT standard PCB, but with foam board in the center of its cross section. A single impact was made to each barrier. Figure 4.20 shows the typical failure mode of the test barriers. The barriers with foam board were observed to have higher impact energy compared to that of the original ODOT PCBs; however, the damage due to cracking, spalling and splashing of the concrete was greater than what was observed for the corresponding ODOT standard barriers. During the first impact, the maximum drop height was 11’-3” which resulted in an impact energy of 31.7 kJ. For the second impact, the maximum drop height was 11’-10” and the resulting impact energy was 33.2 kJ.

Figure 4.20  Failure mode of ODOT standard PCB with foam board: impact side (left) and pavement side (right)
4.3.5 ODOT Standard PCB with Foam Board and Steel Fibers

Impact tests were also performed on ODOT standard PCBs with foam board core reinforced with steel fibers. A single impact was made at the center of the concrete barrier as shown. The typical failure mode for these test barriers is shown in Figure 4.21. Of all the PCB test specimens tested, these PCBs showed the highest energy absorption capacity. During the first impact, the drop height of the Test Rock was 14’-8”, and the resulting impact energy was 41.1 kJ. The drop height was increased to 15’-11” for the second impact test; and the impact energy calculated for the second test was 44.6 kJ.

Figure 4.21  Typical failure mode of ODOT standard PCB with foam board and steel fibers: impact side (left) and pavement side (right)

4.4 Impact Tests on Revised CIP Concrete Barriers

The results from the impact tests performed on the CIP concrete barriers are briefly described in this section. The impact energies, the impact locations and the crack patterns corresponding to each impact location for all the different designs of CIP concrete barriers are presented.

4.4.1 CIP Concrete Barrier with Epoxy Coated Bars and WWF

A total of five impact tests were performed at five locations along the length of barrier CIP-1. The first impact was made at the edge of the rebar section of the concrete barrier. The impact locations and resulting energy for each impact were as follows.
i) Impact near the edge of the right half: The first impact test was performed at a location 5’-3” from the edge of the barrier. The Test Rock was raised to a height of 13’-2” before releasing it to strike the barrier. The resulting impact energy absorbed at this section was 67.9 kJ.

ii) Impact at the center of right half: The second impact was made at a location over the drainage window located at the center of the right half of the barrier. The impact test was performed at this location from a drop height of 17’-5” with an impact energy of 89.7 kJ. Figure 4.22 shows the failure mode of the barrier.

![Figure 4.22](image)

Figure 4.22  CIP-1 after impact at right section, impact side (left) and pavement side (right)

iii) Impact at mid-length of the barrier: The third impact test was performed at the mid-length of the barrier. This section has an overlap of WWF and steel reinforcing bars. The Test Rock was dropped from a height of 22’-7” and this resulted in impact energy of 116.4 kJ. Figure 4.23 shows the failure mode of the test barrier after impact at the section.

![Figure 4.23](image)

Figure 4.23  CIP-1 after impact at center section: impact side (left) and pavement side (right)
iv) Impact at center of the WWF on left half: The fourth impact test was performed at the center of the left half of the barrier. The location of this impact is above the second drainage window located at the center of the WWF section of the barrier. The drop height of the Test Rock was 17’-8” and impact energy delivered at this section was 91.1 kJ. Figure 4.24 shows the failure mode after the impact.

v) Impact near the edge of the WWF section on left half: The final impact on this barrier was made near the edge of the left half of the barrier. The Test Rock was dropped from a height of 14’-7” to deliver an impact energy of 75.2 kJ at this section.

4.4.2 CIP-2 Concrete Barrier with Black Bars and WWF

The second set of impact tests was performed on concrete barrier CIP-2, which had black bars (reinforcing bars without epoxy coating) and plain WWF without epoxy coating. This barrier demonstrated higher impact energy absorption and a better ability to remain intact after an impact than the CIP-1 concrete barrier with epoxy-coated bars and WWF with epoxy coating.

i) Impact near the edge of the right half: The Test Rock was dropped from a height of 19’-10” and the impact location was 4’ from the edge of the right half of the barrier. The resulting impact energy at this section was 102.2 kJ. Figure 4.25 shows the failure mode after the impact.
ii) Impact at the center of the right half: The second impact was made on the barrier above the middle of the first drainage window that is located at the center of the right half. The Test Rock was released from a drop height of 22’ resulting in impact energy of 112.1 kJ. Figure 4.26 shows the failure mode after this impact.

iii) Impact at the center of the barrier: The third impact was made at the center section of the barrier where the longitudinal reinforcing bars and the WWF have an overlap length of 48 inches. The test rock was released from a height of 27’ resulting in impact energy of 128.4 kJ. Figure 4.27 shows the failure mode after the impact.
iv) Impact at the center of the WWF section on the left half: The fourth impact was performed at the center of the WWF section on the left half near the second drainage window. The Test Rock was dropped from a height of 11’-0”. The impact energy delivered by the Test Rock at this section was 113.4 kJ. Figure 4.28 shows the failure mode after the impact.

v) Impact near the edge of the WWF section in the left half: The last impact was made near the edge of the WWF section of the barrier. The Test Rock was released from a maximum height of 19’-8”. The energy delivered at this section was 101.4 kJ.
4.4.3 CIP-3 Concrete Barrier with Epoxy-Coated Bars and WWF Made from Concrete with Steel Fibers

A single impact was made at three sections of CIP-3 concrete barrier with epoxy-coated bars and WWF. This CIP barrier was made from concrete mixed with steel fibers. The first impact was at the center of the right half of the barrier. The second impact was at the center of the barrier where there was overlap between the mesh and the longitudinal reinforcing bars. The last impact was made at the middle of the left half of the barrier, which contained epoxy-coated WWF.

i) Impact in the right half of the barrier: The Test Rock was dropped from a height of 24’-1”. The resulting impact energy delivered to the barrier was 124.1 kJ. Figure 4.29 shows the failure mode after the impact.

![Figure 4.29](image)

Figure 4.29  CIP-3 after impact in the right half: left (impact side) and pavement side (right)

ii) Impact at the mid-length section of the barrier: The second impact test was made at the center of the barrier section, where the WWF and the longitudinal reinforcing bars overlapped. The Test Rock was dropped from a height of 30’-6” delivering impact energy of 157.2 kJ at this section. Figure 4.30 shows the failure mode after the impact.
iii) Impact on the left half: The third and last impact was made on the WWF reinforced section of the barrier above the second drainage window in the left half. The Test Rock was dropped from a height of 24’-6” delivering an impact energy of 126.4 kJ at this section. Figure 4.31 shows the failure mode after the impact.

4.4.4 CIP-4 Concrete Barrier with Epoxy-coated Bars and WWF with PP Fibers

Three impact tests were performed on the concrete barrier with epoxy-coated bars and WWF. CIP-4 test barrier was made from concrete with polypropylene fibers. The first impact was made at the center of the right half of the barrier where the epoxy-coated reinforcing bars were provided, the second impact was made at the center of the barrier
where the longitudinal reinforcing bars overlapped with WWF, and the third impact was made at the center of left half where WWF was provided.

i) Impact on the right half of the barrier: The first impact was made above the first drainage window, which was located at the center of the right half of the barrier. The Test Rock was dropped from a height of 20’-9” which resulted in impact energy of 106.9 kJ. Figure 4.32 shows the failure mode after the impact.

![Figure 4.32  CIP-4 after impact in the right half: impact side (left) and pavement side (right)](image)

ii) Impact at the center of the test barrier: The Test Rock was released from a height of 27’-2” resulting in impact energy of 140 kJ. Figure 4.33 shows the failure mode after the impact.

![Figure 4.33  CIP-4 after impact at center of the barrier: impact side (left) and top view (right)](image)
iii) Impact at the center of left half: The last impact location was above the second drainage hole located at the center of the WWF section of the barrier. The Test Rock was dropped from a height of 20’-6”. This resulted in impact energy of 105.7 kJ at this section of the barrier. Figure 4.34 shows the failure mode after the impact.

![Figure 4.34](image)

Figure 4.34  CIP-4 after impact at WWF section in the left half: impact side (left) and pavement side (right)

4.4.5 CIP-5 Concrete Barrier Deformed WWF with or without Epoxy Coating

The last set of impact tests was performed on a CIP concrete barrier with deformed WWF reinforcement. The right half of the barrier was reinforced with WWF without epoxy coating and the left half with epoxy-coated WWF. The first set of impacts was performed on the right half. Later impacts were made on the left half of the barrier. The energy absorption capacity was generally higher for the WWF without epoxy coating compared to that of the section with epoxy-coated WWF.

i) Impact on the right half: The location of the impact was at the drainage window in the right half of the barrier. The Test Rock was dropped from a height of 15’-4”. The impact energy delivered to the section was 79.2 kJ. Figure 4.35 shows the failure mode after impact. A significant amount of cracking, splashing of concrete and severe crumbling of concrete was observed in this test.
ii) Impact near edge of the right half of the barrier: Two impacts were made at this section. The first impact at this section was made at the extreme end of the right half at a distance of 2’-6” from the right end of the barrier. The drop height for the Test Rock was 11’-3” and the impact energy delivered at the section was 58.2 kJ. The impact was repeated at this section at a level of 1’ above the base of the barrier. The drop height for the Test Rock was 18’-6”. The impact energy delivered by this impact was 95.3 kJ. Figure 4.36 shows the failure mode after impact.

iii) Impact at the left half of the barrier: The drop height for this impact is 21’-0”. The resulting impact energy was 108.3 kJ.
iv) Impact on the mid-section of barrier: A single impact was made at the mid-length of the barrier where the epoxy-coated and non-epoxy-coated WWF overlapped. The drop height of the Test Rock was 23’-1”. The impact energy delivered at the section was 119 kJ. Figure 4.37 shows the failure mode after impact.

![Figure 4.37  CIP-5 after impact at the center section: impact side (left) and top view (right)](image)

v) Impacts at the drainage window in the left half: The Test Rock was dropped from a height of 18”-3”. The resulting impact energy was 94 kJ. Figure 4.38 shows the failure mode after the impact.

![Figure 4.38  CIP-5 after impact at center of WWF epoxy coated section: impact side (left), pavement side (right)](image)

vi) Impact in the left half of the barrier: The impact at this section was made 2’-8” from the left end of the barrier. The Test Rock was dropped from a height of 15’-5”. The impact
energy delivered was 79.5 kJ. Figure 4.39 shows the results after impact.

Figure 4.39  CIP-5 after last impact: edge view (left) and pavement side (right)

A summary of the velocities computed for testing the CIP test barriers is given in Table 4.3 for different impact locations. The corresponding energies developed for each impact are shown in Table 4.4 in kJ, and also in kip-ft in Table 4.5. It is worth recalling that the average maximum energy absorbed by CIP barriers in the 2012 tests with epoxy-coated #5 bars spaced at 18 inches on center was about 55 kJ (41 kip-ft).

Table 4.3   Summary of impact velocities (in feet/sec) for CIP test barriers

<table>
<thead>
<tr>
<th>No.</th>
<th>Unit</th>
<th>Location 1</th>
<th>Location 2</th>
<th>Location 3</th>
<th>Location 4</th>
<th>Location 5</th>
<th>Location 6</th>
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<td>32</td>
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Table 4.4   Summary of impact energies (in kJ) for CIP test barriers

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<th>No.</th>
<th>Unit</th>
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<th>Location 3</th>
<th>Location 4</th>
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Table 4.5  Summary of impact energies in kip-ft for CIP test barriers

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<td>72</td>
<td>90</td>
<td>82</td>
<td>71</td>
<td>60</td>
</tr>
</tbody>
</table>

The improvements obtained by changing the reinforcement details relative to the currently followed ODOT #5 bar details (tested in 2012) are shown in Table 4.6 for each impact location and each test specimen. These percent increases reflect the increase in energy absorption due to the modifications developed in this test program. The average energy absorbed by CIP barrier specimens that were tested in 2012 (55 kJ or 41 kip-ft) was considered as the base energy absorption capacity of CIP concrete barriers for computing the percent increase for the comparison.

Table 4.6  Summary of percent increase in impact energies for CIP barrier test specimens relative to ODOT CIP barriers with #5 bar with 18-inch spacing

<table>
<thead>
<tr>
<th>Unit</th>
<th>Location 1</th>
<th>Location 2</th>
<th>Location 3</th>
<th>Location 4</th>
<th>Location 5</th>
<th>Location 6</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIP-1</td>
<td>26</td>
<td>66</td>
<td>116</td>
<td>69</td>
<td>40</td>
<td>--</td>
<td>Epoxy Coated</td>
</tr>
<tr>
<td>CIP-2</td>
<td>90</td>
<td>108</td>
<td>139</td>
<td>111</td>
<td>89</td>
<td>--</td>
<td>No Epoxy Coating</td>
</tr>
<tr>
<td>CIP-3</td>
<td>--</td>
<td>131</td>
<td>192</td>
<td>135</td>
<td>--</td>
<td>--</td>
<td>With Steel Fiber</td>
</tr>
<tr>
<td>CIP-4</td>
<td>--</td>
<td>99</td>
<td>160</td>
<td>97</td>
<td>--</td>
<td>--</td>
<td>With PP Fiber</td>
</tr>
<tr>
<td>CIP-5</td>
<td>47</td>
<td>76</td>
<td>121</td>
<td>102</td>
<td>75</td>
<td>47</td>
<td>D-7 WWF</td>
</tr>
</tbody>
</table>

Location 3 referred to in Tables 4.3 to 4.6 relate to the energy absorption capacity of each CIP test barrier at mid length. The section at this location has no drainage window. Additionally, the longitudinal reinforcement from the two halves of each test barrier were lap spliced at this location therefore the impact energy recorded at the drainage window is
considered as the governing limit. Considering that a 5 ft. long drainage window is only needed and specified at a spacing of 60 ft. to 90 ft., the governing limit adopted in this study is a conservative limit. However, it may be possible to utilize larger energy absorption capacity at locations without drainage windows than the governing limit corresponding to locations at drainage windows.

4.5 Summary of the Impact Tests on Revised Barriers

The summary of the impact energy absorption capacity of the revised barriers with respect to the current rockfall barriers are described in this section.

4.5.1 Impact Energy Absorption Capacities of Revised PCBs

A summary of the velocities and energies used in the testing of the PCB test specimens is given in Table 4.7. The last column in the table shows the percent increase in energy absorption due to the modifications developed in this test program. The average energy absorbed by Specimens 1 and 2 is considered as the base energy for computing the percent increase for this comparison. The data in the table show that the addition of polypropylene or steel fiber improved the energy absorption capacity of the PCBs. The increase in the maximum capacity of a PCB using the ODOT WWF option with the addition of polypropylene fiber was 12%. Similarly, the maximum increase in the capacity with steel fiber addition recorded was 70%. PCBs with foam board core worked very well and were able to resist impact energy as much as 1.5 times that of the standard ODOT PCB with WWF reinforcement option. However, extensive cracking followed by spalling was observed. The energy absorption was increased by a factor of two relative to the ODOT PCBs when PCB test specimens incorporated foam board core and steel fiber. Figures 4.40 and 4.41 show the performance of different types of PCBs tested in this dissertation.
Table 4.7  Summary of impact velocities and energies for PCBs with modified details

<table>
<thead>
<tr>
<th>No.</th>
<th>Test Unit</th>
<th>Impact Velocity (ft/s)</th>
<th>Impact Energy (kJ)</th>
<th>Impact Energy (kip-ft)</th>
<th>Percent Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>PCB-1</td>
<td>22.2</td>
<td>22</td>
<td>16.5</td>
<td>0</td>
</tr>
<tr>
<td>2</td>
<td>PCB-2</td>
<td>22.7</td>
<td>23</td>
<td>17.3</td>
<td>0</td>
</tr>
<tr>
<td>3</td>
<td>PCB + PP fiber-1</td>
<td>23.5</td>
<td>25</td>
<td>18.5</td>
<td>10</td>
</tr>
<tr>
<td>4</td>
<td>PCB + PP fiber-2</td>
<td>23.7</td>
<td>26</td>
<td>18.9</td>
<td>12</td>
</tr>
<tr>
<td>5</td>
<td>PCB + steel fiber-1</td>
<td>26.6</td>
<td>32</td>
<td>23.7</td>
<td>40</td>
</tr>
<tr>
<td>6</td>
<td>PCB + steel fiber-2</td>
<td>29.2</td>
<td>39</td>
<td>28.7</td>
<td>70</td>
</tr>
<tr>
<td>7</td>
<td>PCB-foam board-1</td>
<td>26.9</td>
<td>33</td>
<td>24.3</td>
<td>44</td>
</tr>
<tr>
<td>8</td>
<td>PCB-foam board-2</td>
<td>27.6</td>
<td>35</td>
<td>25.5</td>
<td>51</td>
</tr>
<tr>
<td>9</td>
<td>PCB-foam + steel-1</td>
<td>30.7</td>
<td>43</td>
<td>31.6</td>
<td>87</td>
</tr>
<tr>
<td>10</td>
<td>PCB-foam + steel-2</td>
<td>32</td>
<td>46</td>
<td>34.2</td>
<td>103</td>
</tr>
</tbody>
</table>

Figure 4.40  Comparison of performance of PCBs with different modifications
4.5.2 Impact Energy Absorption Capacities of Revised CIP Concrete Barriers

The 2012 impact tests revealed that CIP test barriers of 42” height with #5 epoxy-coated bars spaced at 18 inches on center were able to resist an impact energy of 55 to 57.5 kJ (41 to 43 kip-ft) under one-time impact at a location at least 4 feet away from the ends. The energies absorbed at the location of the drainage window using different design alternatives are shown in Figure 4.42. The improvements achieved by using the modified details in this set of impact tests that were conducted in 2013 are shown in Figure 4.43 for the impacts at the drainage window location, which is a weaker section compared to the locations devoid of a drainage widow.

Addition of polypropylene fiber to the concrete used to make the barriers improved the energy absorption capacity by about 100%, while steel fiber addition resulted in an increase in energy absorption capacity by 135% at the drainage window location. The best performance was recorded for the CIP barriers made with concrete containing steel fiber.
Addition of fiber not only increased the energy absorption capacity of barriers but also provided significant crack control and improved structural integrity, possibly offsetting the lack of bond between the reinforcing bars and the surrounding concrete.

Figure 4.42  Summary of energy absorbed by different types of CIP barriers at drainage window location

As evident from the summary of all the results shown in Table 4.6, energy absorption capacity of CIP test barriers was improved by 26% close to the ends to 192% at the locations without drainage windows. The same amounts of impact energies were planned to be delivered at the drainage windows in both halves of each CIP test barrier. This was done to obtain a comparison of the performance of the length with reinforcing bars, with the performance of lengths with WWF. In all the CIP test barriers, the barrier length with #4 reinforcing bars performed far better than the barrier length with WWF for
the same amount of energy delivered in the tests. The extent of damage, as indicated by the extent of crumbling, spalling and spilling of concrete, was much more severe in the barrier lengths with WWF (e.g., Figure 4.35) compared to the damage in the barrier length with reinforcing bars (e.g., Figure 4.32). In some cases, the Test Rock was not fully retained in the sections with WWF. The efforts to tie WWF and keep it in position with correct amount of covers within the forms were also greater than the corresponding efforts in the barrier lengths with reinforcing bars. The WWF also ruptured at the locations of the impacts. Therefore, WWF option with or without epoxy coating did not appear to be suitable for CIP barriers.
If the energy absorption capacity at the drainage window locations is considered as the governing limit and the results corresponding to WWF design are excluded, this improvement in energy absorption capacity is 66% to 130%. The percentages were computed relative to the energy absorption capacity of 55 kJ that was recorded for the control CIP concrete test barriers tested in Phase 1 (with reinforcement details of #5 bars at a spacing of 24 inches on center).

CIP barriers with #4 bars at closer spacing than the corresponding spacing with #5 reinforcing bars using nearly the same amount of steel per unit length provide the most practical design with likely the least impact on cost. The increased energy absorption can be as much as 66% (with epoxy coated bars) to 108% (without epoxy coating) while the cracking performance is also improved due to closer spacing of the bars. Addition of steel or polypropylene fiber will be a suitable option if the CIP concrete barriers are needed in a location where rockfalls with larger impact energies than 91 kJ (with epoxy coated bars) or 115 kJ (without epoxy coating) are expected.
CHAPTER V

YIELD LINE ANALYSIS OF THE ENERGY ABSORPTION CAPACITY
OF CIP CONCRETE BARRIERS AND NUMERICAL MODELING

5.1 Yield Line Theory

Yield line analysis is an approach for determining the ultimate load carrying capacity of reinforced structures mainly slabs and barrier walls (Johansen, 1962). In yield line analysis, a collapse mechanism is determined, with consideration given to the boundary conditions, and it is used in conjunction with the virtual work principle to determine the ultimate load carrying capacity. This method gives us the correct solutions or the upper bound on the ultimate load carrying capacity (Park and Gamble 2000).

In the yield line analysis, the reinforcing bars are assumed to act as wires and the following assumptions are made in the calculation of moments along yield lines.

- the bending stiffness of reinforcing is neglected
- reinforcing bars can develop only tensile stresses
- only bending moments are developed in the slab (no twisting moments)

The ideal ultimate moment of resistance per unit width due to the reinforcement is:

\[ m_u = A_s f_y (d - 0.59 A_s f'_c / f'_c) \]

Where \( A_s \) - area of tension steel per unit width, \( f_y \) - yield strength of the reinforcement, \( d \) - distance from the centroid of the steel to the extreme concrete compression fiber, and \( f'_c \) - compressive strength of the concrete as shown in Figure 5.1.
The following two assumptions are used when computing the resisting moment along the inclined yield line for determining the resisting moment along yield lines. Kinking of rebars is assumed in the Wood’s yield line method (Wood, 1961) whereas in Johansen’s assumptions, the rebars remain straight as shown in Figure 5.2.

\[ M_n = \phi A_s f_y (d - 0.59 A_s \frac{f_y}{f_c}) \]

\[ A_s f_y \]

5.2 AASHTO Yield Line Theory for Concrete Barrier

Hirsch (1978) initiated application of yield line theory to calculate the ultimate strength for concrete barrier. AASHTO LRFD describes the strength of the concrete barrier as shown in Eqns. 5.2 and 5.3. These equations were derived from the yield line theory by equating the external work to the internal work done from Figure 5.3.
\[ L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8H(M_b + M_w)}{M_c}} \]  

\[ R_w = \left(\frac{2}{2L_c - L_t}\right) + \left(8M_b + 8M_w + \frac{M_cL_t^2}{H}\right) \]

Where, \( H \) is the height of barrier, \( L_c \) is the critical length of yield line, \( L_t \) is the longitudinal distribution of impact force \((F_t)\), \( R_w \) is the resistance or ultimate strength of the barrier, \( M_b \) is the flexural resistance of the beam at the top of the barrier, \( M_c \) is the flexural resistance per unit length of the barrier about its longitudinal axis, and \( M_w \) is the flexural resistance of the barrier along its vertical axis.

![Figure 5.3 Equivalent crash load and yield line for concrete barrier (Hirsch, 1978)](image)

5.3 Yield Line Analysis of Concrete Barrier under Impact

To analyze the collapse load under the impact, the yield line pattern was developed from field impact tests and then the virtual work principle is applied. The external work done is calculated by multiplying the external force with the displacement of the force at failure. For the point load \( P \), the external work can be calculated by equation 5.4:

\[ \text{External work} = \int Pdx = F_t \Delta \]

Where \( \Delta \) is the out-of-plane deformation and \( F_t \) is impact force.
The internal work can be calculated along the yield line using the moments and the rotations. The work done by the torsional moments and the shear forces is zero when summed over the entire structure. This is due to the fact that these actions are equal and opposite, on the faces of the same yield line.

For the inclined unit length of yield line at an angle $\alpha$ from vertical, the horizontal rebars and vertical rebars are arranged in a concrete barrier as shown in Figure 5.4.

![Inclined yield line](image)

Figure 5.4  Inclined yield line for a CIP concrete barrier

Therefore, the internal work due to the bending moments is given by equation 5.6:

$$W = \sum_i^1 m_{nu} l_i \theta_{ni} = \sum_i^1 (m_{xu} \cos^2 \alpha + m_{yu} \sin^2 \alpha) l_i \theta_{ni}$$  \hspace{1cm} 5.5

$$W = \sum_i^1 (m_{xu} \theta_{ni} \cos \alpha Y_{oi} + m_{yu} \theta_{ni} \sin \alpha X_{oi}) = \sum_i^1 (m_{xu} \theta_{xi} Y_{oi} + m_{yu} \theta_{yi} X_{oi})$$  \hspace{1cm} 5.6

Where $\theta_{xi}$ and $\theta_{yi}$ are the components of $\theta_{ni}$ in the x- and y-directions respectively, and $X_{oi}$ and $Y_{oi}$ are the projected lengths of the yield line.

For the structures that are reinforced in the x- and y-direction, and the ultimate resisting moment $m_{xu}$ and $m_{yu}$ are known. Therefore, it is easier to deal separately with the x- and y-components of the internal work done. By equating the external work to the internal work, the virtual work equation may be written as

$$F_i \Delta = \sum_i^1 m_{nu} l_i \theta_{ni}$$  \hspace{1cm} 5.7
From the given collapse mechanism the deformations are known and also the moment capacities along the barriers can be calculated. Therefore, the impact energy is computed from the internal energy dissipated along the failure lines.

5.3.1 Failure Mode of the 42” High CIP Concrete Barrier under Rockfall Impact

Three failure modes have been observed in the field. The failure mode in Figure 5.5 is located above the drainage window of 5-ft length and 4-inches high with inclined failure line from the end of drainage window. The moment capacity along this failure plane is calculated on the horizontal projections along horizontal and vertical directions.

![Failure plane above the drainage window](image)

Figure 5.5  Failure plane above the drainage window

The second and the third types of the failure planes are located where there is no drainage window and near the edge of the barrier. These failure planes were triangular in shape as shown in Figure 5.6.

The energy absorption capacity along the failure planes are calculated along the inclined yield lines which have negative moments, $m_h^-$ and $m_v^-$ and along vertical yield lines which has positive moments, $m_h^+$. 

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Since the reinforcement along both faces of the barriers is similar, the negative and positive yield moments are equivalent, i.e. $m^+ = m^- = m_h$ and $m^+_v = m^-_v = m_v$

$$W_T = 2m^-_h \theta_v h + 2m^+_h \theta_v + m^-_v \theta_h L_c = 4m_h \theta_v h + m_v \theta_h L_c$$

$$W_T = 4m_h \left( \frac{2 \Delta}{L_c} \right) h + m_v \left( \frac{\Delta}{h} \right) L_c = 8m_h \frac{\Delta}{L_c} h + m_v \frac{\Delta}{h} L_c$$ \tag{5.8}$$

The moment due to vertical rebars, $m_v$, are calculated at different sections along the barrier heights, starting from the top section of the barrier towards the bottom of the barrier using the following expression.

$$m_v = A_{sv} f_y (d - a/2)$$ \tag{5.9}$$

The moment due to horizontal rebars, $m_h$, are calculated along the sections of the barriers using the following expression.

$$m_h = A_{sh} f_y (d - a/2)$$ \tag{5.10}$$

Where, $A_{sh}$ is the area of horizontal rebars, $A_{sv}$ is the area of vertical rebars, $f_y$ is the yield strength of steel, $d$ is the effective depth, and $a$ is the depth of rectangular compression block.
5.3.2 Reinforcement Detailing of CIP Concrete Barrier

A summary of the reinforcement provided in the CIP concrete barrier tested during 2012 and 2013 is shown in previous Table 5.1. The steel reinforcement has a yield strength of 60 ksi and WWF has a yield strength of 80 ksi. These values were used in the calculation of the ultimate moment capacity.

Table 5.1  Reinforcements provided in CIP concrete test barriers

<table>
<thead>
<tr>
<th>Size</th>
<th>Plain WWR with Epoxy Coating</th>
<th>Plain WWR without Epoxy Coating</th>
<th>Deformed WWR (with or without Epoxy Coating)</th>
<th>Deformed bars (with or without Epoxy Coating)</th>
<th>Deformed bars with Epoxy Coating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barrier section where used</td>
<td>CIP-1 (Left)</td>
<td>CIP-2 (Left)</td>
<td>CIP-3 &amp; 4 (Left), and CIP-5 (Left and Right)</td>
<td>CIP-1 to 4 (Right)</td>
<td>In 2012 Tests (Left and Right)</td>
</tr>
<tr>
<td>Spacing along length</td>
<td>6”</td>
<td>6” (layer 1) and 12” (layer 2)</td>
<td>4”</td>
<td>12”</td>
<td>18”</td>
</tr>
<tr>
<td>Steel area per unit length</td>
<td>0.24 in²</td>
<td>0.21 in²</td>
<td>0.21 in²</td>
<td>0.20 in²</td>
<td>0.20 in²</td>
</tr>
<tr>
<td>Spacing along height</td>
<td>6”</td>
<td>12” (layer 1) and 6” (layer 2)</td>
<td>4”</td>
<td>6.75”</td>
<td>10.5”</td>
</tr>
<tr>
<td>Steel area per unit height</td>
<td>0.24 in²</td>
<td>0.21 in²</td>
<td>0.21 in²</td>
<td>0.34 in²</td>
<td>0.35 in²</td>
</tr>
</tbody>
</table>

The reinforcement areas shown in Table 5.1 are the average areas of steel on either face of the barrier section per one foot length at least 4 feet from the barrier ends. Additionally, #4 inverted U-shaped ties were provided at the top of the barrier section at spacing of 24-inches on center for the revised designs. This reinforcement was mainly provided to minimize cracking, crushing and wedge-shaped concrete failures at the top that were observed in the impact tests conducted in 2012.
5.3.3 Out-of-Plane Deformation Analysis

The high-speed cameras recorded the video of the CIP concrete test barriers from the pavement side of the barriers. The analysis of the videos were performed using the software ARAMIS. This software traces the black dots on the surface of the barriers to compute the strains, cracks and displacements. The following cases are identified for the lateral deformations of the barriers under impact.

Case 1: the impacts on the CIP concrete barriers above drainage window and the resulting maximum out-of-plane displacements are shown in Figure 5.7. The maximum displacement is 104.764-mm (4.1-inch).

![Figure 5.7 Out-of-plane deformations for impact above drainage window](image)

Case 2: The result shown in Figure 5.8 is the results of lateral displacement analysis at the center of the CIP concrete barrier. The barrier is CIP concrete barrier with steel fiber reinforced concrete. The maximum lateral displacement computed at the top section was noted as 129.3-mm (5-inch).
Case 3: the result shown in Figure 5.9 shows the impact near the edge section for the CIP concrete barrier. The maximum out-of-plane deformation at the top section of the barrier is 88.3 mm (3.5-inch).

Figure 5.8 Out-of-plane deformations for impact at center of barrier

Figure 5.9 Out-of-plane deformations for impact near the edge of barrier

5.3.4 Yield Line Analysis of 42” Current CIP Concrete Barrier with Epoxy

The length of the test barrier is 60 ft. and its height is 42 inches with the following reinforcement details. The size of the reinforcing bar used was #5 with diameter of with
0.625-inch and 0.31 square inch in cross-section. The horizontal reinforcement was provided at spacing of 10.5 inch along each face of the barrier and vertical reinforcement was spaced at 2 ft on center. Figure 5.10 shows the rebar spacing along horizontal and vertical direction.

Calculation of the moment capacity of the barrier sections:

![Figure 5.10](image)

Table 5.2 shows the moment capacities of the vertical rebars calculated at various levels, h. Table 5.3 shows the moment capacity due to horizontal rebars. Table 5.4 shows the absorbed energy of the CIP barriers.

**Table 5.2  Moment capacity due to vertical rebars**

<table>
<thead>
<tr>
<th>Height, in</th>
<th>Thickness of cross-section, t – in</th>
<th>Effective depth, d-in</th>
<th>Moment arm, (d-a/2)- in</th>
<th>Moment capacity, k-in/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>12.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>14</td>
<td>14.7</td>
<td>12.4</td>
<td>12.2</td>
<td>91</td>
</tr>
<tr>
<td>23</td>
<td>16.4</td>
<td>14.1</td>
<td>14.0</td>
<td>104</td>
</tr>
<tr>
<td>27</td>
<td>17.1</td>
<td>14.8</td>
<td>14.7</td>
<td>110</td>
</tr>
<tr>
<td>31</td>
<td>17.9</td>
<td>15.6</td>
<td>15.5</td>
<td>115</td>
</tr>
<tr>
<td>35</td>
<td>18.7</td>
<td>16.4</td>
<td>16.2</td>
<td>121</td>
</tr>
<tr>
<td>39</td>
<td>19.4</td>
<td>17.1</td>
<td>17.0</td>
<td>127</td>
</tr>
<tr>
<td>42</td>
<td>20.0</td>
<td>17.7</td>
<td>17.6</td>
<td>131</td>
</tr>
</tbody>
</table>

The average moment due to vertical rebars, $M_{va} = 114$ k-in/ft
Table 5.3  Moment capacity due to horizontal rebars

<table>
<thead>
<tr>
<th>Rebar</th>
<th>Height of rebar – in</th>
<th>Thickness of cross-section, in</th>
<th>Effective depth, d – in</th>
<th>Moment arm, d-a/2-in</th>
<th>Moment capacity, k-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>top</td>
<td>0.0</td>
<td>12.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1 or 2</td>
<td>2.0</td>
<td>12.4</td>
<td>9.4</td>
<td>9.2</td>
<td>137</td>
</tr>
<tr>
<td>3 or 4</td>
<td>12.5</td>
<td>14.4</td>
<td>11.4</td>
<td>11.2</td>
<td>166</td>
</tr>
<tr>
<td>5 or 6</td>
<td>23.0</td>
<td>16.4</td>
<td>13.4</td>
<td>13.2</td>
<td>196</td>
</tr>
<tr>
<td>7 or 8</td>
<td>33.5</td>
<td>18.4</td>
<td>15.4</td>
<td>15.2</td>
<td>226</td>
</tr>
</tbody>
</table>

The average moment due to horizontal rebars per unit height, \( M_{ha} = 207 \text{ k-in/ft} \)

Table 5.4  Energy absorbed by 42” CIP barrier with epoxy coating

<table>
<thead>
<tr>
<th>Failure plane/section</th>
<th>Length of crack, ( L_c ), ft</th>
<th>Depth of crack, ( h ), in</th>
<th>Out of plane deformation, ( \Delta )-in</th>
<th>Average moment, ( m_v ) (kip-in/ft)</th>
<th>Average moment, ( m_h ) (kip-in/ft)</th>
<th>Absorbed Energy (kJ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIP-center section</td>
<td>5</td>
<td>34</td>
<td>4</td>
<td>114</td>
<td>207</td>
<td>43</td>
</tr>
<tr>
<td>CIP-Drainage window</td>
<td>4</td>
<td>38</td>
<td>3</td>
<td>114</td>
<td>207</td>
<td>41</td>
</tr>
<tr>
<td>CIP-edge section</td>
<td>3</td>
<td>26</td>
<td>2.5</td>
<td>114</td>
<td>207</td>
<td>32</td>
</tr>
</tbody>
</table>

5.3.5 Yield Line Analysis of Revised 42” CIP Concrete Barrier without Epoxy

The length of the barriers is 40 ft. and its height is 42 inches with the following reinforcement detailing. The size of the reinforcing bar used was #4 with diameter of 0.5 inch and 0.2 square inch in cross-sectional area. The horizontal reinforcement was provided at a spacing of 6.75 inch along each face of the barrier and vertical reinforcements are spaced at 1 ft on center. Figure 5.11 shows the rebar spacing along horizontal and vertical sections. Table 5.5 shows the moment capacity of the barrier due to the vertical rebars and Table 5.6 shows the moment capacity due to the horizontal rebars. Table 5.7 shows the energy absorption capacity of the CIP concrete barrier.
Figure 5.11  Horizontal and vertical rebars: Cross-section (left) and elevation (right)

Table 5.5  Moment capacity due to vertical rebars

<table>
<thead>
<tr>
<th>Height (h)-in</th>
<th>Thickness of x-section (t)-in</th>
<th>Effective depth d-in,</th>
<th>Moment arm (d-a/2)-in</th>
<th>Moment capacity (Mn)-k.in/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>12</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>14</td>
<td>14.7</td>
<td>12.4</td>
<td>12.3</td>
<td>147</td>
</tr>
<tr>
<td>23</td>
<td>16.4</td>
<td>14.1</td>
<td>14</td>
<td>168</td>
</tr>
<tr>
<td>27</td>
<td>17.1</td>
<td>14.9</td>
<td>14.7</td>
<td>177</td>
</tr>
<tr>
<td>31</td>
<td>17.9</td>
<td>15.7</td>
<td>15.5</td>
<td>186</td>
</tr>
<tr>
<td>35</td>
<td>18.7</td>
<td>16.4</td>
<td>16.3</td>
<td>195</td>
</tr>
<tr>
<td>39</td>
<td>19.4</td>
<td>17.2</td>
<td>17</td>
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</tr>
<tr>
<td>42</td>
<td>20</td>
<td>17.8</td>
<td>17.6</td>
<td>211</td>
</tr>
</tbody>
</table>

The average moment, $M_{va} = 184$ k-in/ft

Table 5.6  Moment capacity due to horizontal rebars

<table>
<thead>
<tr>
<th>Rebar</th>
<th>Heights of rebars–in</th>
<th>Thickness of cross-section, in</th>
<th>Effective depth, d – in</th>
<th>Moment arm, d-a/2-in</th>
<th>Moment capacity, k-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 or 2</td>
<td>3.0</td>
<td>12.6</td>
<td>9.8</td>
<td>9.6</td>
<td>115</td>
</tr>
<tr>
<td>3 or 4</td>
<td>9.8</td>
<td>13.9</td>
<td>11.1</td>
<td>10.9</td>
<td>130</td>
</tr>
<tr>
<td>5 or 6</td>
<td>16.5</td>
<td>15.1</td>
<td>12.4</td>
<td>12.1</td>
<td>146</td>
</tr>
<tr>
<td>7 or 8</td>
<td>23.3</td>
<td>16.4</td>
<td>13.7</td>
<td>13.4</td>
<td>161</td>
</tr>
<tr>
<td>9 or 10</td>
<td>30.0</td>
<td>17.7</td>
<td>15.0</td>
<td>14.7</td>
<td>177</td>
</tr>
<tr>
<td>11 or 12</td>
<td>36.8</td>
<td>19.0</td>
<td>16.3</td>
<td>16.0</td>
<td>192</td>
</tr>
</tbody>
</table>

The average moment per unit height, $M_{hb} = 263$ k-in/ft

124
Table 5.7  Energy absorbed by 42” revised CIP barrier without epoxy coating

<table>
<thead>
<tr>
<th>Failure plane/section</th>
<th>Length of crack, Lc - ft</th>
<th>Depth of crack section, h - in</th>
<th>Out of plane deformation, Δ - in</th>
<th>Average moment, m_v (kip-in/ft)</th>
<th>Average moment, m_h (kip-in/ft)</th>
<th>Absorbed Energy (kJ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIP-center section</td>
<td>5</td>
<td>36</td>
<td>5</td>
<td>184</td>
<td>263</td>
<td>74</td>
</tr>
<tr>
<td>CIP-Drainage window</td>
<td>4</td>
<td>38</td>
<td>4</td>
<td>184</td>
<td>263</td>
<td>71</td>
</tr>
<tr>
<td>CIP-edge section</td>
<td>3.75</td>
<td>30</td>
<td>3.5</td>
<td>184</td>
<td>263</td>
<td>55</td>
</tr>
</tbody>
</table>

5.3.6 Yield Line Analysis of Revised 42” CIP Concrete Barrier with Epoxy

The length of the barriers is 40 ft. and its height is 42 inches with the following reinforcement detailing. The size of the reinforcing bar used was #4 with 0.5 inch diameter and 0.2 square inch in cross-sectional area. The horizontal reinforcement was provided at spacing of 6.75” along each face of the barrier and vertical reinforcements are spaced at 1 ft on center as shown in Figure 5.11. Table 5.8 shows the moment capacity at different levels of the barrier due to vertical reinforcement.

Table 5.8  Moment capacity due to vertical rebars

<table>
<thead>
<tr>
<th>Height, in</th>
<th>Thickness of cross-section, t-in</th>
<th>Effective depth, d-in</th>
<th>Moment arm, d-a/2-in</th>
<th>Moment capacity, k-in/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>12.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>11</td>
<td>14.1</td>
<td>11.8</td>
<td>11.7</td>
<td>112</td>
</tr>
<tr>
<td>16</td>
<td>15.0</td>
<td>12.8</td>
<td>12.7</td>
<td>121</td>
</tr>
<tr>
<td>21</td>
<td>16.0</td>
<td>13.8</td>
<td>13.6</td>
<td>131</td>
</tr>
<tr>
<td>26</td>
<td>17.0</td>
<td>14.7</td>
<td>14.6</td>
<td>140</td>
</tr>
<tr>
<td>31</td>
<td>17.9</td>
<td>15.7</td>
<td>15.5</td>
<td>149</td>
</tr>
<tr>
<td>36</td>
<td>18.9</td>
<td>16.6</td>
<td>16.5</td>
<td>158</td>
</tr>
<tr>
<td>42</td>
<td>20.0</td>
<td>17.8</td>
<td>17.6</td>
<td>169</td>
</tr>
</tbody>
</table>

The average moment, M_va = 122 k-in/ft
Table 5.9 shows the moment capacity due to horizontal rebars and Table 5.10 shows the energy absorption capacity of the barrier.

Table 5.9  Moment capacity due to horizontal rebars

<table>
<thead>
<tr>
<th>Rebar</th>
<th>Heights of rebars-in</th>
<th>Thickness of cross-section, t-in</th>
<th>Effective depth, d, in</th>
<th>Moment arm, d-a/2-in</th>
<th>Moment capacity, k-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 or 2</td>
<td>3.0</td>
<td>12.6</td>
<td>9.8</td>
<td>9.6</td>
<td>92</td>
</tr>
<tr>
<td>3 or 4</td>
<td>9.8</td>
<td>13.9</td>
<td>11.1</td>
<td>10.9</td>
<td>104</td>
</tr>
<tr>
<td>5 or 6</td>
<td>16.5</td>
<td>15.1</td>
<td>12.4</td>
<td>12.1</td>
<td>117</td>
</tr>
<tr>
<td>7 or 8</td>
<td>23.3</td>
<td>16.4</td>
<td>13.7</td>
<td>13.4</td>
<td>129</td>
</tr>
<tr>
<td>9 or 10</td>
<td>30.0</td>
<td>17.7</td>
<td>15.0</td>
<td>14.7</td>
<td>141</td>
</tr>
<tr>
<td>11 or 12</td>
<td>36.8</td>
<td>19.0</td>
<td>16.3</td>
<td>16.0</td>
<td>154</td>
</tr>
</tbody>
</table>

The average moment per unit height, \( M_{ha} = 210 \text{ k-in/ft} \)

Table 5.10  Energy absorbed by 42” revised CIP barrier with epoxy coating

<table>
<thead>
<tr>
<th>Failure plane/section</th>
<th>Length of crack, ( L_c ), ft</th>
<th>Depth of crack, ( h ), in</th>
<th>Out-of-plane deformation, ( \Delta ), in</th>
<th>Average moment, ( m_r ), (kip-in/ft)</th>
<th>Average moment, ( m_h ), (kip-in/ft)</th>
<th>Absorbed Energy, (kJ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIP-center section</td>
<td>5</td>
<td>36</td>
<td>5</td>
<td>122</td>
<td>210</td>
<td>57</td>
</tr>
<tr>
<td>CIP-Drainage window</td>
<td>4</td>
<td>38</td>
<td>4</td>
<td>122</td>
<td>210</td>
<td>56</td>
</tr>
<tr>
<td>CIP-edge section</td>
<td>3.75</td>
<td>30</td>
<td>3.5</td>
<td>122</td>
<td>210</td>
<td>43</td>
</tr>
</tbody>
</table>

5.3.7 Yield Line Analysis of Revised 42” CIP Concrete Barrier of WWF epoxy

The length of the barriers is 40 ft. and its height is 42 inches with the following WWF reinforcement detailing. The amount of areas available per unit length both in vertical and horizontal directions are 0.21 in\(^2\)/ft. Figure 5.12 shows the WWF wire spacing along horizontal and vertical section.
Figure 5.12  D-7 WWF Horizontal and vertical reinforcements

Calculation of the plastic moments due to vertical WWF wires: Table 5.11 shows the moment capacity at different levels of the barrier, h. Table 5.12 shows the moment capacity due to horizontal WWF wires. Table 5.13 shows the energy absorption capacity of the barrier.

Table 5.11  Moment capacity due to vertical WWF wires

<table>
<thead>
<tr>
<th>Height, in</th>
<th>Thickness of cross-section, t, in</th>
<th>Effective depth, d, in</th>
<th>Moment arm, d-a/2</th>
<th>Moment capacity, k-in/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>12.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>14</td>
<td>14.7</td>
<td>12.5</td>
<td>12.3</td>
<td>165</td>
</tr>
<tr>
<td>23</td>
<td>16.4</td>
<td>14.2</td>
<td>14.0</td>
<td>189</td>
</tr>
<tr>
<td>27</td>
<td>17.1</td>
<td>15.0</td>
<td>14.8</td>
<td>199</td>
</tr>
<tr>
<td>31</td>
<td>17.9</td>
<td>15.8</td>
<td>15.5</td>
<td>209</td>
</tr>
<tr>
<td>35</td>
<td>18.7</td>
<td>16.5</td>
<td>16.3</td>
<td>219</td>
</tr>
<tr>
<td>39</td>
<td>19.4</td>
<td>17.3</td>
<td>17.1</td>
<td>229</td>
</tr>
<tr>
<td>42</td>
<td>20.0</td>
<td>17.9</td>
<td>17.6</td>
<td>237</td>
</tr>
</tbody>
</table>

The average moments, $M_{va} = 207$ k-in/ft

Table 5.12  Moment capacity due to horizontal WWF wires

<table>
<thead>
<tr>
<th>WWF/section</th>
<th>Heights of WWF</th>
<th>Thickness of cross-section-in</th>
<th>effective depth, d, in</th>
<th>Moment arm, d-a/2-in</th>
<th>Moment capacity, k-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>0.0</td>
<td>12.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>2.0</td>
<td>12.4</td>
<td>9.9</td>
<td>9.9</td>
<td>133</td>
</tr>
<tr>
<td>22</td>
<td>12.5</td>
<td>14.4</td>
<td>11.9</td>
<td>11.9</td>
<td>160</td>
</tr>
<tr>
<td>32</td>
<td>23.0</td>
<td>16.4</td>
<td>13.9</td>
<td>13.9</td>
<td>186</td>
</tr>
<tr>
<td>42</td>
<td>33.5</td>
<td>18.4</td>
<td>15.9</td>
<td>15.9</td>
<td>213</td>
</tr>
</tbody>
</table>

The average moments per unit height, $M_{ha} = 198$ k-in/ft

127
Table 5.13  Energy absorbed by 42” revised CIP barrier with epoxy coated WWF

<table>
<thead>
<tr>
<th>Failure plane/section</th>
<th>Length of crack, Lc, ft</th>
<th>Depth of crack, h, in</th>
<th>Out-of-plane deformation, Δ, in</th>
<th>Average moment, mv (kip-in/ft)</th>
<th>Average moment, mh (kip-in/ft)</th>
<th>Absorbed Energy (kJ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIP-center section</td>
<td>5</td>
<td>36</td>
<td>5</td>
<td>207</td>
<td>198</td>
<td>61</td>
</tr>
<tr>
<td>CIP-Drainage window</td>
<td>4</td>
<td>38</td>
<td>4</td>
<td>207</td>
<td>198</td>
<td>57</td>
</tr>
<tr>
<td>CIP-edge section</td>
<td>3.75</td>
<td>30</td>
<td>3.5</td>
<td>207</td>
<td>198</td>
<td>45</td>
</tr>
</tbody>
</table>

5.3.8 Yield Line Analysis of Current 50” CIP Concrete Barrier with Epoxy

The size of the reinforcing bar used was #5 with diameter of 0.625 inch and 0.31 square inch in cross-sectional area. The horizontal reinforcement was provided at a spacing of 10.5” along each faces of the barrier and vertical reinforcement was spaced at 2 ft on center. Figure 5.13 shows the rebar spacing along horizontal and vertical directions.

![Figure 5.13](image)

Figure 5.13  Horizontal and vertical rebars: Cross-section (left) and elevation (right)

Calculation of the plastic moments due to vertical rebars:

Table 5.14 shows the moment capacity due to the vertical rebars.
Table 5.14  Moment capacity due to vertical rebars

<table>
<thead>
<tr>
<th>Height, in</th>
<th>Thickness of cross-section, t – in</th>
<th>Effective depth, d-in</th>
<th>Moment arm, (d-a/2)- in</th>
<th>Moment, k-in/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>12.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>14</td>
<td>14.7</td>
<td>12.4</td>
<td>12.2</td>
<td>91</td>
</tr>
<tr>
<td>23</td>
<td>16.4</td>
<td>14.1</td>
<td>14.0</td>
<td>104</td>
</tr>
<tr>
<td>27</td>
<td>17.1</td>
<td>14.8</td>
<td>14.7</td>
<td>109</td>
</tr>
<tr>
<td>31</td>
<td>17.9</td>
<td>15.6</td>
<td>15.5</td>
<td>115</td>
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<tr>
<td>35</td>
<td>18.7</td>
<td>16.4</td>
<td>16.2</td>
<td>121</td>
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<td>19.4</td>
<td>17.1</td>
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<td>126</td>
</tr>
<tr>
<td>42</td>
<td>20.0</td>
<td>17.7</td>
<td>17.6</td>
<td>131</td>
</tr>
<tr>
<td>46</td>
<td>20.8</td>
<td>18.4</td>
<td>18.3</td>
<td>136</td>
</tr>
<tr>
<td>50</td>
<td>21.5</td>
<td>19.2</td>
<td>19.1</td>
<td>142</td>
</tr>
</tbody>
</table>

The average moments, M_{va} = 120 k-in/ft

Calculation of the plastic moments due to horizontal rebars:

Table 5.15 shows the moment capacity due to the horizontal rebars. Table 5.16 shows the energy absorption capacity of the barrier.

Table 5.15  Moment capacity due to horizontal rebars

<table>
<thead>
<tr>
<th>Rebar</th>
<th>Heights of rebars</th>
<th>Thickness of cross-section, in</th>
<th>Effective depth, d – in</th>
<th>Moment arm, d-a/2-in</th>
<th>Moment capacity, k-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>top</td>
<td>0.0</td>
<td>12.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>1 or 2</td>
<td>2.0</td>
<td>12.4</td>
<td>9.4</td>
<td>9.2</td>
<td>136</td>
</tr>
<tr>
<td>3 or 4</td>
<td>12.5</td>
<td>14.4</td>
<td>11.4</td>
<td>11.2</td>
<td>166</td>
</tr>
<tr>
<td>5 or 6</td>
<td>23.0</td>
<td>16.4</td>
<td>13.4</td>
<td>13.2</td>
<td>196</td>
</tr>
<tr>
<td>7 or 8</td>
<td>33.5</td>
<td>18.4</td>
<td>15.4</td>
<td>15.2</td>
<td>226</td>
</tr>
<tr>
<td>9 or 10</td>
<td>44.0</td>
<td>20.8</td>
<td>17.8</td>
<td>17.8</td>
<td>265</td>
</tr>
</tbody>
</table>

The average moments, M_{ha} = 237 k-in/ft
Table 5.16  Energy absorbed by 50” CIP barrier

<table>
<thead>
<tr>
<th>Failure plane</th>
<th>Length of crack, Lc, ft</th>
<th>Depth of crack, h, in</th>
<th>Out of plane deformation Δ, in</th>
<th>Average moment, mv (kip-in/ft)</th>
<th>Average moment, mh (kip-in/ft)</th>
<th>Absorbed Energy (kJ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIP-center section</td>
<td>6</td>
<td>44</td>
<td>5</td>
<td>120</td>
<td>237</td>
<td>61</td>
</tr>
<tr>
<td>CIP-Drainage window</td>
<td>5</td>
<td>46</td>
<td>4</td>
<td>120</td>
<td>237</td>
<td>56</td>
</tr>
<tr>
<td>CIP-edge section</td>
<td>4</td>
<td>36</td>
<td>3</td>
<td>120</td>
<td>237</td>
<td>41</td>
</tr>
</tbody>
</table>

5.5.9 Yield Line Analysis of Revised 50” CIP Concrete Barrier with Epoxy

The size of the reinforcing bar used was #4 with diameter of 0.5 inch and 0.2 square inch in cross-sectional area. The horizontal reinforcement was provided at spacing of 6.75” along each face of the barrier and vertical reinforcement was spaced at 1 ft on center. Figure 5.14 shows the rebar spacing along horizontal and vertical directions.

![Diagram of rebar spacing](image)

Figure 5.14  Horizontal and vertical rebars: Cross-section (left) and elevation (right)

Calculation of the plastic moments due to vertical rebars:

Table 5.17 shows the moment capacity due to the vertical rebars. Table 5.18 shows the moment capacity due to the horizontal rebars. Table 5.19 shows the impact energy absorption capacity of the barrier.
Table 5.17  Moment capacity due to vertical rebars

<table>
<thead>
<tr>
<th>Height, in</th>
<th>Thickness of cross-section, t - in</th>
<th>Effective depth, d-in</th>
<th>Moment arm, (d-a/2)- in</th>
<th>Moment capacity, k-in/ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>12.0</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>11</td>
<td>14.1</td>
<td>11.8</td>
<td>11.7</td>
<td>112</td>
</tr>
<tr>
<td>16</td>
<td>15.0</td>
<td>12.8</td>
<td>12.7</td>
<td>121</td>
</tr>
<tr>
<td>21</td>
<td>16.0</td>
<td>13.8</td>
<td>13.6</td>
<td>131</td>
</tr>
<tr>
<td>26</td>
<td>17.0</td>
<td>14.7</td>
<td>14.6</td>
<td>140</td>
</tr>
<tr>
<td>31</td>
<td>17.9</td>
<td>15.7</td>
<td>15.5</td>
<td>149</td>
</tr>
<tr>
<td>36</td>
<td>18.9</td>
<td>16.6</td>
<td>16.5</td>
<td>158</td>
</tr>
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<td>42</td>
<td>20.0</td>
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<td>169</td>
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<tr>
<td>48</td>
<td>21.1</td>
<td>18.9</td>
<td>18.7</td>
<td>180</td>
</tr>
</tbody>
</table>

The average moment, \( M_{va} = 145 \text{ k-in/ft} \)

Table 5.18  Moment capacity due to horizontal rebars

<table>
<thead>
<tr>
<th>Rebar</th>
<th>Heights of rebars</th>
<th>Thickness of cross-section, t-in</th>
<th>Effective depth, d-in</th>
<th>Moment arm, d-a/2-in</th>
<th>Moment capacity, k-in</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 or 2</td>
<td>3.0</td>
<td>12.6</td>
<td>9.8</td>
<td>9.6</td>
<td>92</td>
</tr>
<tr>
<td>3 or 4</td>
<td>9.8</td>
<td>13.9</td>
<td>11.1</td>
<td>10.9</td>
<td>104</td>
</tr>
<tr>
<td>5 or 6</td>
<td>16.5</td>
<td>15.1</td>
<td>12.4</td>
<td>12.1</td>
<td>117</td>
</tr>
<tr>
<td>7 or 8</td>
<td>23.3</td>
<td>16.4</td>
<td>13.7</td>
<td>13.4</td>
<td>129</td>
</tr>
<tr>
<td>9 or 10</td>
<td>30.0</td>
<td>17.7</td>
<td>15.0</td>
<td>14.7</td>
<td>141</td>
</tr>
<tr>
<td>11 or 12</td>
<td>36.8</td>
<td>19.0</td>
<td>16.3</td>
<td>16.0</td>
<td>154</td>
</tr>
<tr>
<td>13 or 14</td>
<td>43.5</td>
<td>20.3</td>
<td>17.5</td>
<td>17.3</td>
<td>166</td>
</tr>
</tbody>
</table>

The average moments, \( M_{ha} = 217 \text{ k-in/ft} \)

Table 5.19  Energy absorbed by revised 50” CIP barrier

<table>
<thead>
<tr>
<th>Failure plane</th>
<th>Length of crack, ( L_c ), ft</th>
<th>Depth of crack, ( h ), in</th>
<th>Out of plane deformation, ( \Delta ), in</th>
<th>Average moment, ( m_v ), (kip-in/ft)</th>
<th>Average moment, ( m_h ), (kip-in/ft)</th>
<th>Absorbed Energy (kJ)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CIP-center section</td>
<td>6</td>
<td>44</td>
<td>6</td>
<td>145</td>
<td>217</td>
<td>73</td>
</tr>
<tr>
<td>CIP-Drainage window</td>
<td>5</td>
<td>46</td>
<td>5</td>
<td>145</td>
<td>217</td>
<td>71</td>
</tr>
<tr>
<td>CIP-edge section</td>
<td>4</td>
<td>36</td>
<td>4</td>
<td>145</td>
<td>217</td>
<td>54</td>
</tr>
</tbody>
</table>
5.4 Summary of the Impact Energy Absorption by CIP Concrete Barriers

The impact energy absorption capacity for the 50” CIP concrete barriers with epoxy and without epoxy coated bars were predicted from the tests on 42” CIP concrete barriers performed during both phases of impact tests and the corresponding yield line analysis. The yield line analysis was performed for the current 50-inch CIP concrete barrier and revised CIP concrete barriers; and these results were used to predict the maximum energy absorption capacity for each of the designs.

5.4.1 Impact energy of the 42” and 50” CIP concrete barrier with epoxy coating

Table 5.20 lists the impact energy absorption capacity of 42” CIP concrete barriers from yield line analysis and tests. The predicted energy absorption capacity of 50” CIP concrete barrier was calculated from the test results obtained from 42” CIP concrete barrier multiplied by the percentage increase in the analysis of yield line for 50” CIP concrete barriers.

The percentage increase in the impact energy absorption from the yield line analysis for the current 50” CIP concrete barrier is calculated as factor, $r_c$:

$$r_{cc} = \frac{61-43}{43} = 0.42$$

The percentage increase in the impact energy absorption from the yield line analysis for the revised 50” CIP concrete barrier is calculated as factors, $r_c$ for impact at center section, $r_d$ for impact above drainage window, and $r_e$ for impact at the edge section:

$$r_c = \frac{73-57}{57} = 0.28$$

$$r_d = \frac{71-56}{56} = 0.27$$

132
These factors were used to predict the impact energy absorption capacity of 50” CIP concrete barriers.

Table 5.20  Energy absorption capacity of 42” CIP concrete barrier with epoxy coating

<table>
<thead>
<tr>
<th>Impact Location</th>
<th>Current 42” CIP concrete barrier - epoxy coated</th>
<th>Revised 42” CIP concrete barrier with epoxy coating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Analysis</td>
<td>Tests</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Center section</td>
<td>43 kJ</td>
<td>55 kJ</td>
</tr>
<tr>
<td>Drainage window</td>
<td>41 kJ</td>
<td>-</td>
</tr>
<tr>
<td>Edge section</td>
<td>32 Kj</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 5.21 lists the impact energy absorption of 50” CIP concrete barrier projected from the analysis and predicted values.

Table 5.21  Predicted impact energy absorption of 50” CIP concrete barrier with epoxy coating

<table>
<thead>
<tr>
<th>Impact Location</th>
<th>Current 50” CIP concrete barrier - epoxy coated</th>
<th>Revised 50” CIP concrete barrier with epoxy coating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Analysis</td>
<td>Predicted</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Center section</td>
<td>61 kJ</td>
<td>78 kJ</td>
</tr>
<tr>
<td>Drainage window</td>
<td>56 Kj</td>
<td>-</td>
</tr>
<tr>
<td>Edge section</td>
<td>41 Kj</td>
<td>-</td>
</tr>
</tbody>
</table>
5.4.2 Impact Energy of the Concrete Barrier without Epoxy Coating

Table 5.22 lists the impact energy absorption capacity of revised 42” CIP concrete barriers from yield line analysis and tests.

Table 5.22 Energy absorbed by 42” CIP concrete barrier without epoxy coating

<table>
<thead>
<tr>
<th>Impact Location</th>
<th>Revised 42” CIP concrete barrier without epoxy coating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Analysis</td>
</tr>
<tr>
<td>Center section</td>
<td>74 kJ</td>
</tr>
<tr>
<td>Drainage window</td>
<td>71 kJ</td>
</tr>
<tr>
<td>Edge section</td>
<td>55 kJ</td>
</tr>
</tbody>
</table>

Table 5.23 lists the impact energy absorption capacity of revised 50” CIP concrete barriers projected from yield line analysis and tests. The impact energy absorption capacity of 50” CIP was predicted in similar manner as described for 50” CIP concrete barrier with epoxy coated bars.

Table 5.23 Predicted impact energy absorption of 50” CIP concrete barrier without epoxy coating

<table>
<thead>
<tr>
<th>Impact Location</th>
<th>Revised 50” CIP concrete barrier without epoxy coating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Analysis</td>
</tr>
<tr>
<td>Center section</td>
<td>91 kJ</td>
</tr>
<tr>
<td>Drainage window</td>
<td>88 kJ</td>
</tr>
<tr>
<td>Edge section</td>
<td>68 kJ</td>
</tr>
</tbody>
</table>
5.5 Finite Element Analysis of the Field Impact Tests using Abaqus/Explicit

The concrete barrier and impacting Steel Ball were modeled as shown in the Figure 5.15. The height of the CIP concrete barrier is 42 inch with a length of 40 ft and the size of the Steel Ball is 24 inch in diameter. The CIP concrete barrier was modeled using C3D8R elements of concrete and Test Rock using C3D10M elements of steel in Abaqus. The reinforcement of the concrete is modelled with beam elements displayed in Figure (ABAQUS elements B31).

The beam elements are coupled with the elements of the concrete barrier with the *EMBEDDED ELEMENTS function of ABAQUS. With this function the nodes of a beam element are constrained to the nodes of the solid element in which it is located. This means that the displacement of the node of the beam element is an average value of the displacements of the neighboring nodes of the solid element in which the beam element is embedded.

![Concrete barrier and impacting rocks (left), and reinforcing bars (right)](image)

Figure 5.15  Concrete barrier and impacting rocks (left), and reinforcing bars (right)

Table 5.24 summarizes the type and number of elements, the number of nodes and the number of degrees of freedom (dof) for each component: concrete barrier, reinforcing bars, and Steel Ball.
Table 5.24 Element type and number of elements, nodes and dof for each component.

<table>
<thead>
<tr>
<th>Component</th>
<th>FEM type</th>
<th>No. of elements</th>
<th>No. nodes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete barrier</td>
<td>C3D8R</td>
<td>66744</td>
<td>75189</td>
</tr>
<tr>
<td>Rebar</td>
<td>B31</td>
<td>2717</td>
<td>2751</td>
</tr>
<tr>
<td>Steel balls</td>
<td>C3D10M</td>
<td>2562</td>
<td>3946</td>
</tr>
</tbody>
</table>

Table 5.25 shows the basic material properties used for the concrete, the reinforcement and the Steel ball. The reinforcement and impacting balls are made of mild

Table 5.25 Basic material properties used for the analyses.

<table>
<thead>
<tr>
<th>Material</th>
<th>Mass density</th>
<th>Elastic Modulus</th>
<th>Poisson’s ratio</th>
<th>Comp stress</th>
<th>Ten stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>0.000225 lb-s²/in⁴</td>
<td>3604996 psi</td>
<td>0.18</td>
<td>4000 psi</td>
<td>435 psi</td>
</tr>
<tr>
<td>Steel</td>
<td>0.00073 lb-s²/in⁴</td>
<td>29000000 psi</td>
<td>0.30</td>
<td>60000 psi</td>
<td>60000 psi</td>
</tr>
</tbody>
</table>

5.5.1 Brittle Cracking Model

This is one of the constitutive models for concrete available in ABAQUS/Explicit to model brittle materials like concrete. This model is designed for cases where the overall material behavior is dominated by tensile cracking. It assumes that the compressive behavior is always linear elastic, which does not resemble reality and is a main weakness of the model.

It is most accurate in applications where the brittle behavior dominates such that the assumption that the material is always linear elastic in compression is adequate. On the other hand the Brittle Cracking Model allows the removal of elements based on a brittle failure criterion avoiding in theory large distortions of elements. Figure 5.16 displays the stress-strain-curve for the Brittle Cracking Model as used for the analyses (Martin, 2010).
Figure 5.16  Stress-strain-curve for the Brittle Cracking Model for Concrete.

The brittle cracking model detects failure using Rankine failure criterion and identifies failure in tension. Before failure, the material is linear elastic in tension too.

5.5.2 Concrete Damage Plasticity

This material model is also available for concrete models in Abaqus/Explicit. The Concrete Damaged Plasticity Model uses the concept of isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity to represent the inelastic behavior of concrete. In contrast to the Brittle Cracking Model it allows the definition of strain hardening in compression and can be defined to be sensitive to the straining rate, which resembles the behavior of concrete more realistically (Martin, 2010).

In contrast to the Brittle Cracking Model the Concrete Damaged Plasticity Model does not contain a failure criterion and thus does not allow the removal of elements during the analyses.

Figures 5.17 and 5.18 show the material properties of the concrete based on concrete class B50 (Jankowiak and Lodygowski, 2005).
Figure 5.17  Stress-strain-curve for concrete under compression loads.

Figure 5.18  Stress-strain-curve for concrete under tension.

5.5.3 Boundary Conditions and Initial Velocities

The concrete barrier is supported along its base in the vertical direction. The horizontal restraint is provided along the back edge of the barrier simulating the support provided by the pavement.
The range of initial velocity values ranges from relatively slow 10 ft/s to 30 ft/s. The ranges of the impact velocity considered the practical velocity obtained from the field impact tests. The initial velocities are applied to the center node of the impacting Steel Ball.

The general contact interaction between concrete barrier and impacting Steel Ball was used including all the inner surfaces of the concrete barriers.

5.6 ARAMIS Application

These are optical measuring technique employed by Trilion Quality Systems. They develops precision 3D optical measurement and inspection testing devices. Data produced with their systems may be used for full-field measurement of 3D deformation and strain, to validate finite element models, illustrate material modes of failure.

ARAMIS is an industrial grade solution providing all necessary functionalities even for complex research tasks. The dynamic measuring system ARAMIS provides for any number of measurement of points information about:

- 3D Coordinates
- 3D Displacements
- Deformation
- Speed
- Acceleration

These results are presented in versatile manners and are available to export as ASCII datasets. Due to a visualization of the recorded camera images combined with diagrams, the component behavior can be analyzed easily and in an intuitive manner.
5.7 FEA Comparison to ARAMIS

The results from the ARAMIS can be compared to the results from FEA. Hence, ARAMIS is native import of FEA data-sets (ABAQUS, LS-DYNA, PAM-STAMP, AutoForm and ASCII). Full-field comparison of geometry, deformation, displacements and strain values Calculation and comparison of major and minor strain, thickness reduction, 3D shape/geometry, X, Y, Z displacements, and etc., can be established.

5.7.1 Out-of Plane Displacements

The CIP concrete barriers shown below has the revised steel reinforcements and impacting above the drainage opening. From the finite element modeling of concrete, the maximum lateral displacements were 2.95 inches (74.93 mm) as shown in Figure 5.19.

![Image](image)

**Figure 5.19** Out of plane deformations, Abaqus

From the ARAMIS analysis, the maximum lateral displacements were 2.25 inches (57 mm) as shown in Figure 5.20. These results are in a reasonable accuracy within 30% accuracy of the field displacement analyzed in ARAMIS.
5.7.2 Strains

The validation of the finite element modeling of strains on the concrete barriers were made with ARAMIS. The maximum strains from finite element modeling is 0.024 as shown in Figure 5.21.

The maximum strains from ARAMIS 0.0261 as shown in Figure 5.22. Typical results for the precast concrete barriers and CIP concrete barriers are shown in the appendices at the end of this dissertation.
The strain result from the finite element modeling using Abaqus and the strain result from the high speed camera analysis using ARAMIS matches reasonably very well. Typical results at various stages for the ARAMIS analysis are shown in the Appendices. The finite element models can be used for further analysis of the impact problems without conducting expensive field impact tests.

Figure 5.22 Strains, ARAMIS
CHAPTER VI
CONCLUSION AND RECOMMENDATION

6.1 Introduction

Rockfalls along a slope can reach the roadway at the bottom and create a hazard to the roadway users. Placement of 32” high precast concrete barriers (PCBs) next to the roadway or construction of 42” or 50” cast-in-place (CIP) barriers along the edge of the pavement (at the shoulder of the road) are two common solutions practiced in Ohio for the protection of roadway users against falling rocks. This chapter provides the conclusions drawn from the test results obtained from impact tests for concrete barriers with current ODOT details and revised reinforcement details. The design modifications developed in this project for PCBs include the use of polypropylene or steel fiber in concrete. A new PCB design with foam board core was also developed and evaluated with and without the addition of steel fiber. In the case of CIP barriers, the use of smaller diameter bars at closer spacing, or the use of WWF (welded wire fabric) with or without the addition of polypropylene or steel fiber to concrete were also evaluated.

Repeated low energy impacts at the same location cause significant cracking and weakening of concrete barriers, leading to a reduction in energy absorption relative to the energy absorption due to a single high-energy impact event. Concrete barriers can withstand much larger impact energy if no prior impact had occurred at the same location. The practical implication of this finding is that it is preferable to consider replacement of
PCBs immediately after a rockfall has impacted the barrier, even if it was a low-energy impact with minimum visible cracking. Similarly, it is also preferred to repair or replace the portion of a CIP barrier that was subjected to a low-energy rockfall with no visible signs of failure.

The cubic shaped rocks tend to have larger contract times and were able to rotate about the line of contact upon impact thereby making several contacts under one impact. The test barriers performed better under impacts from cubic shaped rocks than the corresponding spherical shaped rocks or rocks with rounded surfaces for same amount of impact energies. The cubic shapes represent natural rocks more closely than the rounded shapes. Therefore, the energy values obtained from the impact tests conducted in this project with spherical shaped balls are expected to be lower bound values; meaning that the barriers are expected to be able to absorb larger amounts of impact energy than what is recorded from the impact tests in this dissertation.

6.2 Precast Concrete Barriers

Based on the impact tests performed in this project, the following conclusions may be made regarding PCBs:

1. The stability of PCBs against later loading is primarily provided by its own self-weight, friction at the base, tipping of the barrier about the toe, and the rotation of bolted hinged joints between adjacent units. Under a high energy of impact, the PCB tends to tip about the toe and slide toward the roadways particularly, when the impact occurs at the joint between units.

2. PCBs made according to the current ODOT details and specifications are suitable for maximum impact energy of up to 24 kJ (17.9 kip-ft). Severe spalling and crumbling
of concrete happens in such barriers under impact loading.

3. The use of steel or polypropylene fibers increased the energy absorption capacity of the barriers under impact. The revised designs showed significant improvement in terms of the absorption of the impact energy, and also the failure patterns with reduced cracking and spalling. Addition of 7.5 lb/yd$^3$ polypropylene fiber to concrete used for making ODOT standard PCBs can increase the energy absorption to 26 kJ (18.9 kip-ft). Similarly, the addition of 40 lb/yd$^3$ steel fiber can increase the energy absorption to 39 kJ (28.7 kip-ft). When a 2 inch thick foam board is provided in the core of ODOT standard PCB, the energy absorption can increase to 37 kJ (25.5 kip-ft); but the shattering of concrete may make this option unsuitable for practical usage. When steel fiber of 40 lb/yd$^3$ is added to the concrete with foam core option, the energy absorption can be increase to 46 kJ (34.2 kip-ft). A summary of the allowable impact energies for PCBs are summarized in Table 6.1.

4. PCBs with no internal reinforcement possess negligible energy absorption capacity and are unsuitable for rockfall protection

<table>
<thead>
<tr>
<th>Test Unit</th>
<th>Impact Energy (kJ)</th>
<th>Impact Energy (kip-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PCBs with current ODOT details (WWF option)</td>
<td>24</td>
<td>17.9</td>
</tr>
<tr>
<td>PCB with 7.5 lb/yd$^3$ of polypropylene fiber</td>
<td>26</td>
<td>18.9</td>
</tr>
<tr>
<td>PCB with 40 lb/yd$^3$ of steel fiber</td>
<td>39</td>
<td>28.7</td>
</tr>
<tr>
<td>PCB with foam board core</td>
<td>37</td>
<td>25.5</td>
</tr>
<tr>
<td>PCB with foam and 40 lb/yd$^3$ of steel fiber</td>
<td>46</td>
<td>34.2</td>
</tr>
</tbody>
</table>
6.3 Cast-in-Place (CIP) Concrete Barriers

Based on the impact tests performed in this project, the following conclusions may be drawn regarding CIP concrete barriers:

1. CIP barriers of 42” height made according to the current ODOT details and specifications are able to resist impact energy up to 57.5 kJ (42.6 kip-ft). Severe spalling and crumbling of concrete happens under impact loading on such CIP concrete barriers.

2. Severe lack of bond between epoxy-coated bars and the surrounding concrete under impact loading is a serious problem under rockfall impact.

3. By reducing the diameter of the reinforcing bars and reducing the bar spacing both vertically and horizontally, the energy absorption capacity of the barriers can be increased significantly.

4. The use of polypropylene or steel fiber in concrete improves the energy absorption capacity of CIP concrete barriers by as much as 66 to 130% at the drainage window. The use of fiber also reduces cracking, spalling and splashing of concrete.

5. Welded wire mesh (WWF) is not recommended as a steel reinforcement option for CIP concrete barriers due to practical reasons and rupturing of wires under impact loading. The energy absorption capacities were not included in the summary table given below (Table 6.2).

Table 6.2 shows a summary of the impact energy absorption capacity of 42” and 50” CIP concrete barriers at drainage windows and are conservative relative to the locations where there is no drainage window. These energy values may be considered for design purposes.
Table 6.2 Summary of energy absorption capacity of CIP concrete barriers

<table>
<thead>
<tr>
<th>Barrier Height</th>
<th>Description</th>
<th>Maximum Allowed Energy</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>kJ</td>
</tr>
<tr>
<td>42”</td>
<td>ODOT current design with #5 at larger spacing with ECB</td>
<td>55</td>
</tr>
<tr>
<td></td>
<td>Revised ODOT #4 at closer spacing with ECB</td>
<td>91</td>
</tr>
<tr>
<td></td>
<td>Revised ODOT #4 at closer spacing with ECB and 7.5 lb/yd³ polypropylene fibers</td>
<td>109</td>
</tr>
<tr>
<td></td>
<td>Revised ODOT #4 at closer spacing with ECB and 40 lb/yd³ steel fibers</td>
<td>127</td>
</tr>
<tr>
<td></td>
<td>Revised ODOT #4 at closer spacing with black bars (no epoxy coating)</td>
<td>115</td>
</tr>
<tr>
<td>50”</td>
<td>ODOT current design with #5 at larger spacing with ECB</td>
<td>78</td>
</tr>
<tr>
<td></td>
<td>Revised ODOT #4 at closer spacing with ECB</td>
<td>116</td>
</tr>
<tr>
<td></td>
<td>Revised ODOT #4 at closer spacing with ECB and 7.5 lb/yd³ polypropylene fibers</td>
<td>138</td>
</tr>
<tr>
<td></td>
<td>Revised ODOT #4 at closer spacing with ECB and 40 lb/yd³ steel fibers</td>
<td>161</td>
</tr>
<tr>
<td></td>
<td>Revised ODOT #4 at closer spacing with black bars (no epoxy coating)</td>
<td>143</td>
</tr>
</tbody>
</table>

6.4 Future Recommendations

Different shapes of the concrete barriers for impact against rock impact needs to be developed with full-scale impact tests and computer simulations. As the computer and software facilities are becoming more powerful, both concrete and steel together with the bond will be modeled in accurately. The following recommendations will be made for further research both in field tests and computer simulation.

- Impact tests on different designs including shapes, and different boundary conditions will be required for further investigations.
• Sections of concrete barriers need to be studied with provision of different layers which helps in energy absorption capacity.

• Numerous concrete barriers need to be tested under progressive impact energy with single impacts made to each barriers

• The bond of the rebars to the concrete should be studied in detail under impacts

• The effect of different shear reinforcements in the concrete barriers needs to be investigated under impact loads.

• The finite element modeling of reinforced concrete structures and the bond of steel to concrete need to be enhanced which will improve the simulation of impact tests.
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ASTM A 185 Specification for Steel Welded Wire, Plain, for Concrete Reinforcement.

ASTM A 497 Specification for Steel Welded Wire, Deformed, for Concrete Reinforcement.

Structural Specialty Conference of the Canadian Society for Civil Engineering, Montréal, Canada, 10 pp.


Jankowiak and Lodygowski (2005): Identification of parameters of concrete Damage plasticity constitutive model, Poznan University of Technology, Institute of Structural Engineering (ISE)


APPENDIX A

TYPICAL ACCELEROMETER DATA

Accelerometer results on Test Rocks and typical PCB and CIP Concrete Barriers

Accelerometer on Test Rock and PCBs

Figure A1 Accelerometer on Test Rock

Figure A2 Accelerometer on PCB
Figure A3  Accelerometer on Test Rock

Figure A4  Accelerometer on PCB

Figure A5  Accelerometer on Test Rock with CIP concrete barrier against concrete pavement
Figure A6  Accelerometer on CIP concrete barrier against concrete pavement

Figure A7  Accelerometer on Test Rock with CIP concrete barrier against concrete pavement

Figure A8  Accelerometer on CIP concrete barrier against concrete pavement
Figure A9  Accelerometer on Test Rock with CIP concrete barrier against asphalt pavement

Figure A10  Accelerometer on CIP concrete barrier against asphalt pavement

Figure A11  Accelerometer on Test Rock with CIP concrete barrier against asphalt pavement
Figure A12  Accelerometer on CIP concrete barrier against asphalt pavement
APPENDIX B

TYPICAL ARAMIS ANALYSIS STRAIN STAGES

Strain analysis from ARAMIS for PCB and CIP Concrete Barrier

I) Precast Concrete Barriers

(a)  
(b)  
(c)  
(d)
II) Cast-in-place Concrete Barriers

The results from the test on the CIP concrete barriers were analysed from high speed camera and it is shown in the Figure B1.
Figure B2  1st hit: rebar section-center, 91 kJ: a) stage 1-major; b) stage 1-minor; c) stage 20-minor; d) stage 20-major; e) stage 40-major; f) stage 40-minor; g) stage 60-minor; h) stage 60-major; i) stage 80-major; j) stage 80-minor; k) stage 100-minor; l) stage 100-major
APPENDIX C

YIELD LINE ANALYSIS

A) Energy Absorption of CIP by Yield Line Theory Current 42" ODOT CIP with epoxy coating

Calculation of the ultimate moments

a) Vertical rebars

\[
\begin{align*}
\text{Diameter of #5 rebars, } d &= 0.625 \text{ in} \\
\text{Area of rebars, } As &= 0.31 \text{ in}^2 \\
\text{Spacing of rebars, } s &= 2 \text{ ft} \\
\text{Area steel per unit ft, } &= 0.16 \text{ in}^2/\text{ft} \\
\text{Com. strength of concrete, } f'c &= 4 \text{ ksi} \\
\text{Yield strength of steel, } fy &= 60 \text{ ksi}
\end{align*}
\]

Depth of neutral axis, \( a \)

\[
a = \beta e = \frac{A_j f_j}{0.85 f'_c x b} = \frac{0.155 \times 60}{0.85 \times 4 \times 12} = 0.228 \text{ in/ft}
\]

Moment capacity of vertical rebars at various level from the top of 42" concrete barrier are shown in Table 5.2.

b) Horizontal rebars

\[
\begin{align*}
\text{Diameter of #5 rebars, } d &= 0.625 \text{ in} \\
\text{Area of rebars, } As &= 0.31 \text{ in}^2 \\
\text{Spacing of rebars, } s &= 10.5 \text{ in} \\
\text{Com. strength of concrete, } f'c &= 4 \text{ ksi} \\
\text{Yield strength of steel, } fy &= 60 \text{ ksi} \\
\text{No. of rebars in one layer, } n &= 4
\end{align*}
\]

Depth of neutral axis, \( a \)

\[
a = \beta e = \frac{A_j f_j}{0.85 f'_c x b} = \frac{4 \times 0.31 \times 60}{0.85 \times 4 \times 12} = 0.521 \text{ in}
\]

Moment capacity due to horizontal rebars are shown in Table 5.3.
the rotational components along the horizontal and vertical directions are calculated in terms of the deflection, $\Delta$

$$\Theta_h = \frac{\Delta}{h} \quad \text{and} \quad \Theta_v = \frac{2\Delta}{L_c}$$

The absorbed energy along the vertical and two inclined yield lines are

$$W_T = 2m_h \Theta_v h + mh^+2\Theta_v + mv^+\Theta_h L_c$$

$$W_T = 4m_h \Theta_v h + mv \Theta_h L_c$$

since $mh^-$ and $mh^+$ are equivalent;

$$W_T = 4m_h (2\Delta/L_c) h + mv (\Delta/h) L_c$$

$$W_T = 8m_h \Delta/L_c h + mv \Delta/h L_c$$

The energy absorbed capacity is shown in Table 5.4

**B) Energy Absorption of CIP by Yield Line Theory Revised 42" CIP without epoxy coating**

calculation of the ultimate moments

a) Vertical rebars

<table>
<thead>
<tr>
<th>Diameter of #4 rebars, d=</th>
<th>0.5</th>
<th>in</th>
</tr>
</thead>
<tbody>
<tr>
<td>Area of rebars, As=</td>
<td>0.2</td>
<td>in$^2$</td>
</tr>
<tr>
<td>Spacing of rebars, s=</td>
<td>1</td>
<td>ft</td>
</tr>
<tr>
<td>Area steel per unit ft,</td>
<td>0.2</td>
<td>in$^2$/ft</td>
</tr>
<tr>
<td>Com. strength of concrete,</td>
<td>f'c=</td>
<td>4</td>
</tr>
<tr>
<td>Yield strength of steel,</td>
<td>fy=</td>
<td>60</td>
</tr>
</tbody>
</table>

Depth of neutral axis, $a$

$$a = \beta c = \frac{A_s x f_y}{0.85 x f' c x b} = \frac{0.2 x 60}{0.85 x 4 x 12} = 0.294 \text{ in/ft}$$
b) Horizontal rebar

- Diameter of #4 rebars, \( d = 0.5 \) in
- Area of rebars, \( A_s = 0.2 \) in\(^2\)
- Spacing of rebars, \( s = 6.75 \) in
- Comp. strength of concrete, \( f'c = 4 \) ksi
- Yield strength of steel, \( f_y = 60 \) ksi
- Number of rebars in one layer, \( n = 6 \)
- Total area provided, on each side, \( A_{st} = 1.2 \)

Depth of neutral axis, \( a = 0.50 \) in

Moments of horizontal rebars is shown in Table 5.6 and the energy absorption capacity is shown in Table 5.7

C) Energy Absorption of CIP by Yield Line Theory_Revised CIP-Epoxy coated bars

Calculation of the ultimate moments

a) Vertical rebars

- Diameter of #4 rebars, \( d = 0.5 \) in
- Area of rebars, \( A_s = 0.2 \) in\(^2\)
- Spacing of rebars, \( s = 1 \) ft
- Area steel per unit ft, \( 0.2 \) in\(^2/ft\)
- Comp. strength of concrete, \( f'c = 4 \) ksi
- Yield strength of steel, \( f_y = 60 \) ksi

Depth of neutral axis, \( a = 0.294 \) in/ft

Moment capacity of vertical rebars at various level from the top of 42" concrete barrier is shown in Table 5.8.
b) Horizontal rebar

Diameter of #4 rebars, \( d = 0.5 \) in
Area of rebars, \( A_s = 0.2 \) in²
Spacing of rebars, \( s = 6.75 \) in
Comp. strength of concrete, \( f'_c = 4 \) ksi
Yield strength of steel, \( f_y = 60 \) ksi
Number of rebars in one layer, \( n = 6 \)
Total area, on each side, \( A_{st} = 1.2 \) in²

Depth of neutral axis, \( a \)

\[
a = \beta c = \frac{A_s f'_c}{0.85 f'_c b} = \frac{6 \times 0.2 \times 60}{0.85 \times 4 \times 42} = 0.50 \text{ in}
\]

Moments of horizontal rebars is shown in Table 5.9 and the energy absorption capacity is given in Table 5.10.

D) Energy Absorption of CIP by Yield Line Theory Current 42” ODOT CIP-WWF

calculation of the plastic moments

a) Vertical WWF

Diameter of D7 WWF, \( d = 0.298 \) in
Area of WWF, \( A_s = 0.21 \) in²/unit length
Spacing of rebars, \( s = 4 \) in
Comp. strength of concrete, \( f'_c = 4 \) ksi
Yield strength of steel, \( f_y = 80 \) ksi

Depth of neutral axis, \( a \)

\[
a = \beta c = \frac{A_s f'_c}{0.85 f'_c b} = \frac{0.21 \times 4 \times 80}{0.85 \times 4 \times 42} = 0.412 \text{ in/ft}
\]

Moment capacity of vertical rebars at various level from the top of 42” concrete barrier, is shown in Table 5.11.
b) Horizontal WWF

Diameter of D7 WWF, \( d = 0.298 \) in  
Area of WWF, \( A_s = 0.21 \) in\(^2/ft\)  
Spacing of rebars, \( s = 4 \) in  
Comp. strength of concrete, \( f'_c = 4 \) ksi  
Yield strength of steel, \( f_y = 80 \) ksi

Depth of neutral axis, \( a \)

\[
a = \beta c = \frac{A_s f_y}{0.85 f'_c b} = 0.118 \text{ in}
\]

Moments of horizontal rebars is shown in Table 5.12 and energy absorption capacity is shown in Table 5.13.

**E) Energy Absorption of CIP by Yield Line Theory Current 50" ODOT CIP with epoxy coating**

calculation of the ultimate moments

a) Vertical rebars

Diameter of #5 rebars, \( d = 0.625 \) in  
Area of rebars, \( A_s = 0.31 \) in\(^2\)  
Spacing of rebars, \( s = 2 \) ft  
Area steel per unit ft, \( A_{st} = 0.155 \) in\(^2/ft\)  
Comp. strength of concrete, \( f'_c = 4 \) ksi  
Yield strength of steel, \( f_y = 60 \) ksi

Depth of neutral axis, \( a \)

\[
a = \beta c = \frac{A_s f'_c}{0.85 f'_c x b} = \frac{0.155 \times 60}{0.85 \times 4 \times 12} = 0.228 \text{ in/ft}
\]

Moment capacity of vertical rebars at various level from the top of 50" concrete barrier is shown in Table 5.14.

b) Horizontal rebars

Diameter of #5 rebars, \( d = 0.625 \) in  
Area of rebars, \( A_s = 0.31 \) in\(^2\)
Spacing of rebars, s = 10.5 in
Comp. strength of concrete, f'c = 4 ksi
Yield strength of steel, fy = 60 ksi
Number of rebars in one layer, n = 5

Depth of neutral axis, a

\[
a = \beta c = \frac{A_s f_y}{0.85 f'c x b} = \frac{5 \times 0.31 \times 60}{0.85 \times 4 \times 50} = 0.547 \text{ in}
\]

Moments of horizontal rebars is given in Table 5.15 and the energy absorption capacity is given in Table 5.16.

**F) Energy Absorption of CIP by Yield Line Theory Revised 50” CIP epoxy**

calculation of the ultimate moments

a) Vertical rebars

Diameter of #4 rebars, d = 0.5 in
Area of rebars, As = 0.2 in²
Spacing of rebars, s = 1 ft
Area steel per unit ft, = 0.2 in²/ft
Comp. strength of concrete, f'c = 4 ksi
Yield strength of steel, fy = 60 ksi

Depth of neutral axis, a

\[
a = \beta c = \frac{A_s f_y}{0.85 f'c x b} = \frac{0.2 \times 60}{0.85 \times 4 \times 12} = 0.294 \text{ in/ft}
\]

Moment capacity of vertical rebars at various level from the top of 50” concrete barrier is shown in Table 5.17.

b) Horizontal rebar

Diameter of #5 rebars, d = 0.5 in
Area of rebars, As = 0.2 in²
Spacing of rebars, s = 10.5 in
Comp. strength of concrete, f'c = 4 ksi
Yield strength of steel, fy = 60 ksi
Number of rebars in one layer, n = 7
Depth of neutral axis, $a$

$$a = \beta e = \frac{A_z x f_z}{0.85 x f' c' x b} = \frac{7 \times 0.20 \times 60}{0.85 \times 4 \times 50} = 0.49 \text{ in}$$

Moments of horizontal rebars is shown in Table 5.18 and the energy absorption capacity is shown in Table 5.19.