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Prototyping and Field Testing of a Demand-Responsive Rumble Strip Mechanism

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Disclaimer

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16. Abstract

This project involved the development, design, prototyping and testing of a Demand-Responsive Transverse Rumble Strip (DRTRS) mechanism, which becomes active (lowers an array of strips) only when necessary in order to alert drivers of downstream risks. Three alternative mechanisms for the deployment of DRTRS designed and evaluated. A first prototype using an electric actuator was tested. Results illustrate the vibration and noise generated by the prototype. Our evaluation concluded that an innovative hydraulically-activated design is the best approach to deploy the DRTRS. The proposed DRTRS apparatus is modular, and the mechanical components of the DRTRS units are reliable with few components. The hydraulic system will need regular maintenance. However, this system is placed in a cabinet outside of the travel lanes. The DRTRS deployment cost is comparable to existing solutions for intersections, school zones, toll lanes, and speed control zones. A second prototype based on the hydraulic system was built and it is about to be tested on a public facility at the University of Nevada Las Vegas. Results from this testing will provide information about its effectiveness and potential insights to further improve our design to make it even more cost and safety effective.

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Executive Summary

This project involved the development, design, prototyping and testing of a Demand-Responsive Transverse Rumble Strip (DRTRS) mechanism, which becomes active (lowers an array of strips) only when necessary in order to alert drivers of downstream risks. Various studies indicate that noise level increases of 4 dB or greater are sufficient to alert drivers using transverse rumble strips [1], [2], [3]. Ideally, the DRTRS would be installed on travel lanes upstream of locations with traffic safety concerns or where safety improvements are required. The DRTRS can be used as a standalone safety improvement or in conjunction with other improvements, such as railroad crossing arms or flashing beacons in order to make them more effective.

Existing practice involves the use of permanent transverse rumble strips, which are either milled in or installed above the pavement using synthetic materials. Given that rumbles are always there (active), drivers get used to them and the surprise effect diminishes over time. Hence, permanent transverse rumble strips eventually lose effectiveness. In addition, frequent contact with rumble strips produces unnecessary vehicle deterioration, discomfort, noise, and pavement wear. Because of unnecessary noise and discomfort, permanent milled rumble strips have limited use, whereas DRTRS will have a much broader range of applications, such as in school zones, in residential areas, or on highways. Moreover, the DRTRS will be useful in controlling speeds along facilities of special interest such as urban parks, commercial zones, hospitals, or sites with high-crash frequency.

The DRTRS will be active only when needed, preventing drivers from becoming accustomed to the rumble strip effects, while minimizing noise, vehicle deterioration, and wear of, as compared to permanent rumble strips. Hence, drivers' attention will be regained to address distractions, low visibility, or fatigue, as well as to reduce speed along special zones, such as animal crossings or facilities with high-crash frequencies due to excessive speeds. The DRTRS may be activated through push buttons, traffic controllers, and/or detection systems. Additionally, the DRTRS provides redundancy in the case of autonomous vehicles in order to minimize the likelihood of crashes because of failures and/or malfunctions in the detection or navigation systems. The DRTRS does this by offering an alternative communication channel that can alert the autonomous vehicle to slow down or stop, just as it does with human drivers. The DRTRS has the potential to address at least one of the six research priorities in the Fixing America's Surface Transportation (FAST) Act: promoting safety.

This study designed and evaluated three alternative mechanisms for the deployment of DRTRS. A first prototype using an electric actuator was tested. Results illustrate the vibration and noise generated by the prototype. Our evaluation concluded that an innovative hydraulically-activated design is the best approach to deploy the DRTRS. The proposed DRTRS apparatus is modular, and the mechanical components of the DRTRS units are reliable with few components. The hydraulic system will need regular maintenance. However, this system is placed in a cabinet outside of the travel lanes. The DRTRS deployment cost is comparable to existing solutions for intersections, school zones, toll lanes, and speed control zones. A second prototype based on the hydraulic system was built and it is about to be tested on a public facility at the University of Nevada Las Vegas. Results from this testing will provide information about its effectiveness and potential insights to further improve our design to make it even more cost and safety effective.

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INTRODUCTION

Problem Statement

Promoting safety is one of the six research priorities in the Fixing America's Surface Transportation (FAST) Act as well as a priority for many state agencies. According to the National Safety Council (NSC), in 2016, as many as 40,000 people died in the U.S. in motor vehicles crashes[1]. This represents a 6% rise from 2015 and a 14% increase in deaths since 2014. In 2015, 5,376 pedestrians and 818 bicyclists died in crashes with motor vehicles (National Highway Traffic Safety Administration, Traffic Safety Facts)[2]. This corresponds to 17.7% of the 35,092 total U.S. crash fatalities that year. Millions more are seriously injured. The cost of motor-vehicle deaths, injuries, and property damage in 2016 was estimated at $432 billion, a 12% increase from 2015.

There is consensus that human errors are the most significant contributors to the occurrence and severity of a crash. Visual signals are by far the most common approach to alert drivers about the need to slow down, pay attention, or stop. However, distractions, fatigue and/or low visibility warrant pursuing alternative mechanisms to engage acoustic and haptic senses to regain drivers' attention [4]. Highway zones near transit stops represent a continuous traffic safety risk due to high pedestrian activity and the rush of users to catch the next bus, which often requires crossing multiple travel lanes. This risk is particularly relevant to low-income users, who regularly depend on transit service, and western metro regions with very wide highways. At bus stops there is also the high risk of cars crashing into the bus or each other. According to the Federal Transit Administration, we have observed an increase in transit fatalities of 37% from 2007 to 2016[3]. In the United States, mechanisms such as transverse rumble strips are mainly used on approaches to intersections, toll plazas, horizontal curves, and work zones to slow down traffic [4]. Traditional transverse rumble strips show effectiveness [5] [6] [7]. However, drivers tend to become familiar with their locations over time and their effectiveness diminishes. In addition, some drivers try to avoid the static rumbles [4], creating hazardous driving conditions. Unnecessary noise, vehicle deterioration, and pavement wear are additional concerns associated with traditional transverse rumble strips.

Literature Review

Multiple agencies have studied the effectiveness of rumble strips. The NCHRP Synthesis of Highway Practice 191 (Use of Rumble Strips to Enhance Safety) reported a crash reduction of 14% to 100% from 10 before-and-after studies that investigated the effectiveness of transverse rumble strips [8]. A study by the Federal Highway Administration [9] investigated the safety effect of transverse rumble strips on approaches to stop-controlled intersections using the Empirical Bayes method. Results indicated that transverse rumble strips may be effective in reducing severe injury crashes at minor road stop-controlled intersections. However, an increase occurred in property damage-only crashes. It was not possible to determine the reasons for this tradeoff. A limited economic analysis indicated a reduction in crash harm of about $6,600 per intersection per year due to the installation of transverse rumble strips.

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[1] http://www.nsc.org/NewsDocuments/2017/12-month-estimates.pdf

[2] https://crashstats.nhtsa.dot.gov/Api/Public/ViewPublication/812375

[3] https://www.transit.dot.gov/sites/fta.dot.gov/files/docs/ntd/66016/2016-ntst-appendix.pdf

The Texas Department of Transportation (DOT) utilizes transverse rumble strips at various locations, including high-speed signalized intersections with sight restrictions or high-crash rates, and at newly installed stop- or signal-controlled intersections [5]. The Maryland DOT recommends transverse rumble strips on approaches to signalized intersections where there is a safety problem, where other warning devices have failed to reduce crash frequency (in facilities without adequate stopping sight distance or sufficient visibility of signals or signs), and on intersections at unexpected locations [6].

To analyze the effect of transverse rumble strips on drivers' behavior, the Minnesota DOT completed a series of studies. The first study used a driving simulator to investigate driver-stopping performance. The results showed that transverse rumble strips make drivers use their brakes more and apply them earlier [7]. The second study focused on sleep-deprived drivers and showed positive results. A third study revealed that after the first set of transverse rumble strips in real-world approaches, drivers slowed down earlier, compared to locations without treatment; on average, the difference was 2.0 to 5.0 mph [10].

The Western Transportation Institute documented the current practice among transportation agencies and proposed guidelines regarding the design, installation, and use of rumble strips [11]. Yang et al. [4] studied the impact of transverse thermoplastic rumble strips in terms of the sound and vibration drivers feel inside the vehicle, their choice of speed, and their braking behavior when approaching an intersection. The levels of stimuli experienced by drivers were measured using a sound level meter and an accelerometer to measure acceleration rates along the longitudinal, lateral, and gravitational axes. Speeds were measured with a radar gun with ±0.1 mph accuracy and radar. Video data provided vehicle braking, swerving, or shifting maneuvers. Five sites in Alabama were used in this study. Although the study provided excellent results and insights, it was limited to a single vehicle and thermoplastic rumble strips. The study also highlighted that various previous studies have provided inconsistent results regarding the speed effect of transverse rumble strips. Hence, further comprehensive evaluations are required on the basis of these inconsistencies and the NCHRP study [8], Guidelines for Selection of Speed Reduction Treatments at High-Speed Intersections. Our proposed solution is likely to address some of the issues associated with transverse rumble strips and provide superior benefits. However, research is required to determine the right configuration for variable operational conditions.

PROPOSED SOLUTION

Design Overview

We proposed to retrofit roadway zones upstream of bus stops, transit stations, pedestrian crossings, and any other area with traffic safety concerns with a Demand-Responsive Transverse Rumble Strip (DRTRS) mechanism, which becomes active (lowers an array of strips) only when necessary to alert drivers of downstream risks. Multiple alternatives were considered and evaluated for the detail design, prototyping and testing of the DRTRS. This report describes the top three alternatives. The first alternative was designed, prototyped and tested. The second alternative was considered and evaluated but was not designed because its prototyping requires a special customized component which is not easy to build. The third alternative was designed, and it is currently under prototyping and testing. The idea behind the DRTRS is to provide the roadway profile illustrated in Figure 1 only when required. Figure 2 depicts a potential deployment setup for an approach with three lanes and a crosswalk.

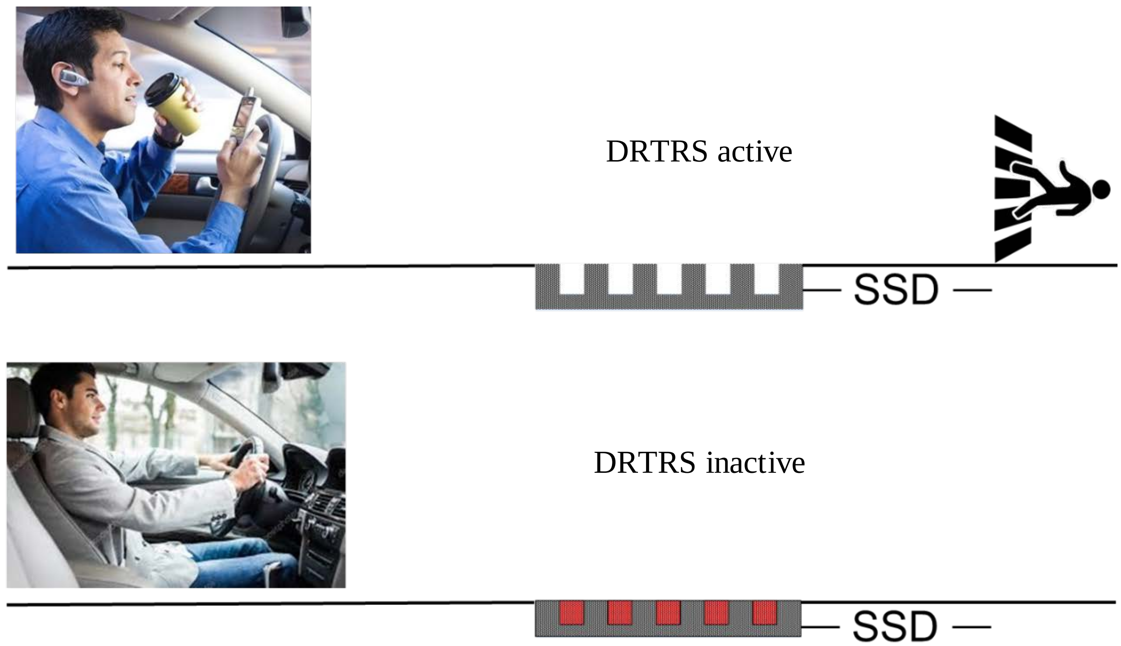
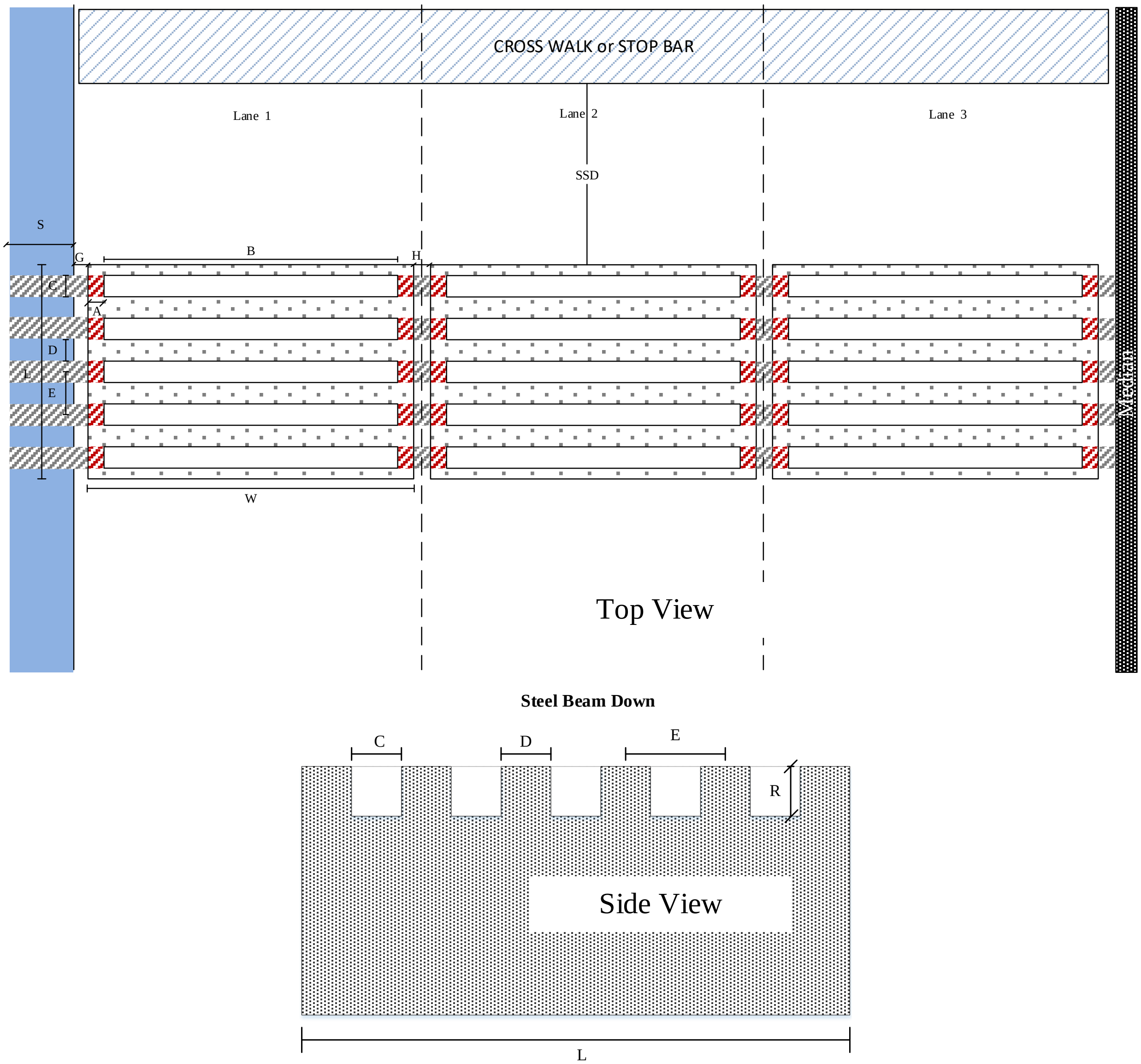


Figure 1. DRTRS Concept.

First Design and Prototype

The first alternative for the implementation of the DRTRS consists of a concrete box frame that houses an array of rectangular box beams, Figure 3. Each DRTRS unit can be lowered or raised by a roller assembly mechanism, Figure 4. The beams are spaced at regular distances from each other and are transverse to the flow of vehicular traffic. When the beams are raised, the concrete box frame together with the beams provide a flat rolling surface flush with the level of the roadway. When the beams are lowered, the holes in the concrete box create transverse rumble strips.

The proposed modular design is compact, less than 10-inches high, and robust (few components). It was developed to maintain functionality under various conditions including severe temperature, as well as variations and existence of rain, snow, and dirt. This design can be quickly installed and uninstalled. The current design enables setting the depth of the rumbles at two different levels.



A= Distance from the edge of the housing box to a rumble

B= Length of a DRTRS => min = 10 ft – 2*A; Max = 12 ft – 2*A

C= Width of a DRTRS = 4 inches

D= Gap between two DRTRS = 8 inches

E= Center to center distance between two DRTRS = 12 inches

G= Distance from the left lane mark to the housing box

H= Distance between two adjacent housing boxes

S= Shoulder with

L= Length of the housing box for a set of DRTRS

W= Width of the housing box for a set of DRTRS

SSD = Stopping sight distance

Permanent rumble strips on the housing box

Permanent rumble strips on the pavement and shoulder

R = Maximum depth of the rumbles = 0.5 inches

Figure 2. A Potential Deployment Setup

Figure 3. Schematic of the DRTRS in the Field

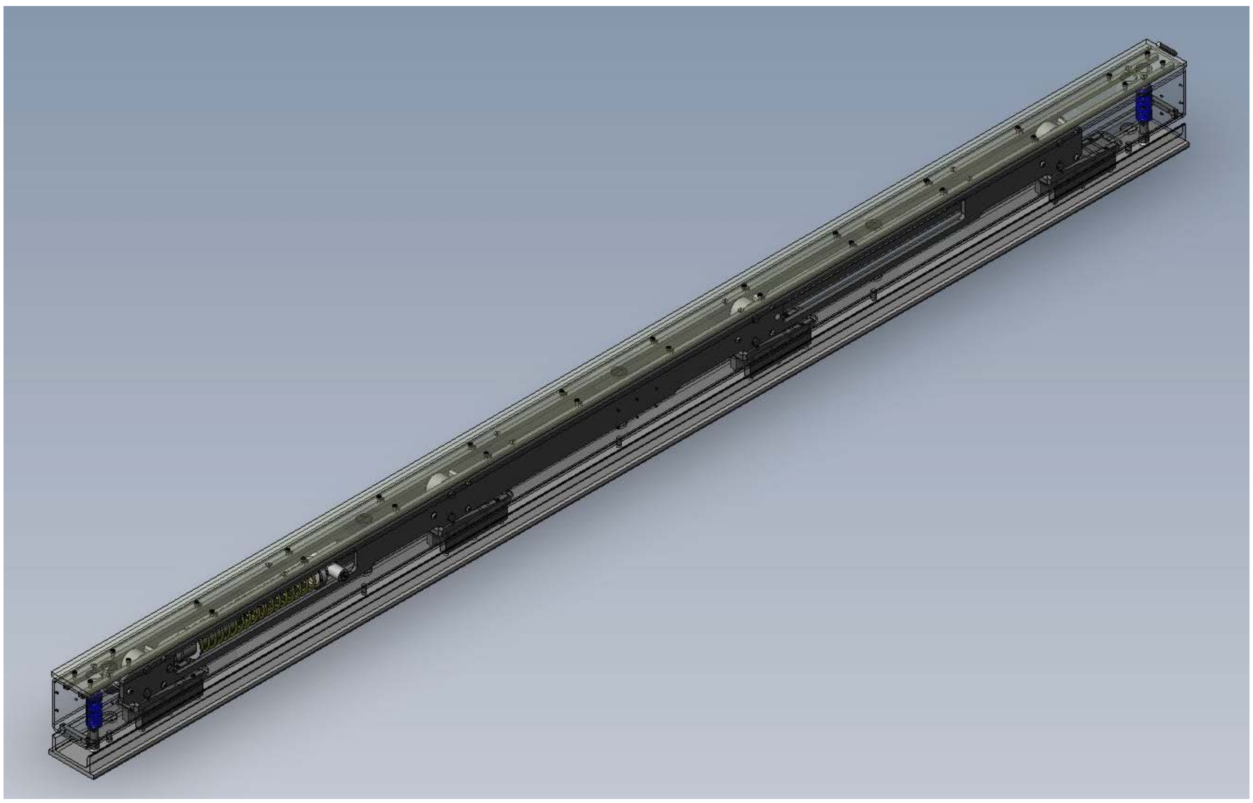


Figure 4. A Unit Box Beam and Roller Assembly of the DRTRS

Figure 5 represents a zoomed view of the most important components of the box beam and roller assembly units within the DRTRS. Key components include:

1) Top plate that aligns with the road when at the highest level
2) Rectangular box beam
3) Shock absorber
4) Lifting mechanism consisting of: (i) a linear actuator and (ii) a stepped cam and follower subsystem that allows lifting the rumble strip to the desired height.

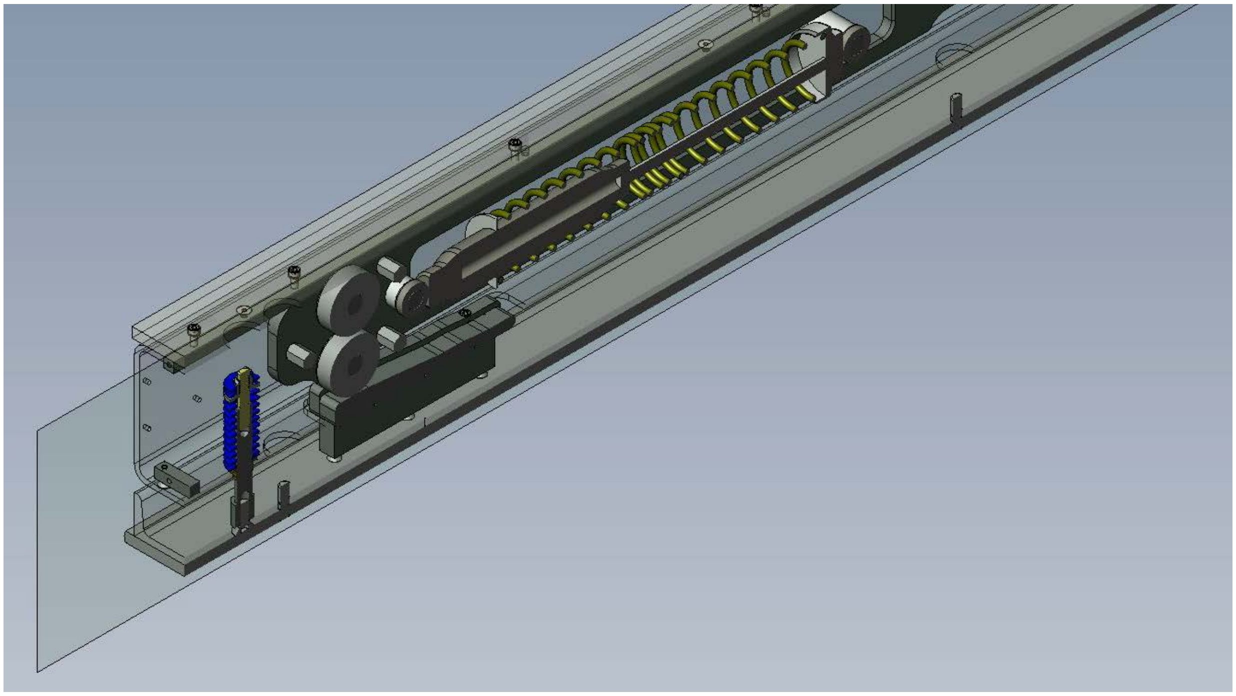


Figure 5. A Zoomed View (Sectioned) Showing the Most Important Unit Elements of the DRTRS

General Operations Description:

Initially, the wearing plate is flush with the top of the concrete box and roadway. To lower the unit, the actuator retracts, causing the shock absorbers to compress the rolling assembly to roll down, lowering the box beam. To return to the initial position, the actuator expands, and the shock absorbers push the box beam, causing the top plate to become flush with the top of the concrete box.

Second Design and Prototype

Figure 6 illustrates the alternative that was considered for the implementation of the DRTRS. Figure 7 shows the three major components of the DRTRS from top to bottom: (i) a Wearing Plate, (ii) a Rumbles Assembly, and (iii) a Base Plate. The Base Plate will be bound to the pavement and is designed to provide housing and support to the Rumbles Assembly and Wearing Plate. In addition, the Base Plate provides connection to the conduits. A cross section of the Rumbles Assembly and Wearing Plate for a single rumble is provided in Figure 3. The Rumble Plates, housed in the Rumbles Assembly, can be lowered or raised by adding or removing fluid from the Hydraulic Lifting Tube. The Rumble Plates are spaced at regular distances from each other and are transverse to the flow of vehicular traffic. When the plates are raised, together with the Wearing Plate, they provide a flat rolling surface flush with the level of the roadway. When the Rumble Plates are lowered, the holes in the Wearing Plate create transverse rumble strips.

The proposed modular design is compact, five and a half inches high, and robust (few components). It is designed to maintain functionality under various conditions including severe temperature, as well as variations and existence of rain, snow, and dirt. The spaces between the Rumbles Frame and the Rumble Plate, which create the rumble effect, will be sealed using gaskets to ensure that water, snow, or dirt will not penetrate the Rumbles Assembly. By housing the Rumbles Assembly in the Base Plate, the entire mechanism, and any sensors, can be quickly removed and replaced. Maintenance of the rumbles and any sensor can be done offsite, away from traffic. As a potential desirable future capability, pressure from the traffic on the Rumbles Assembly can be used to count, classify vehicles and measure speeds.

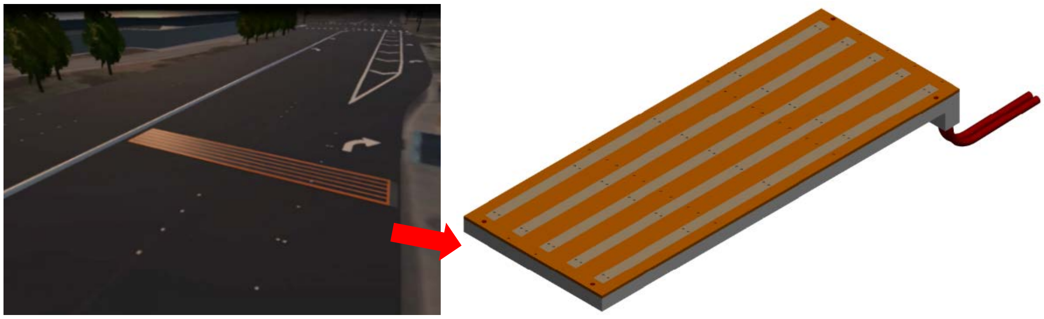


Figure 6. Schematic of the DRTRS in the Field

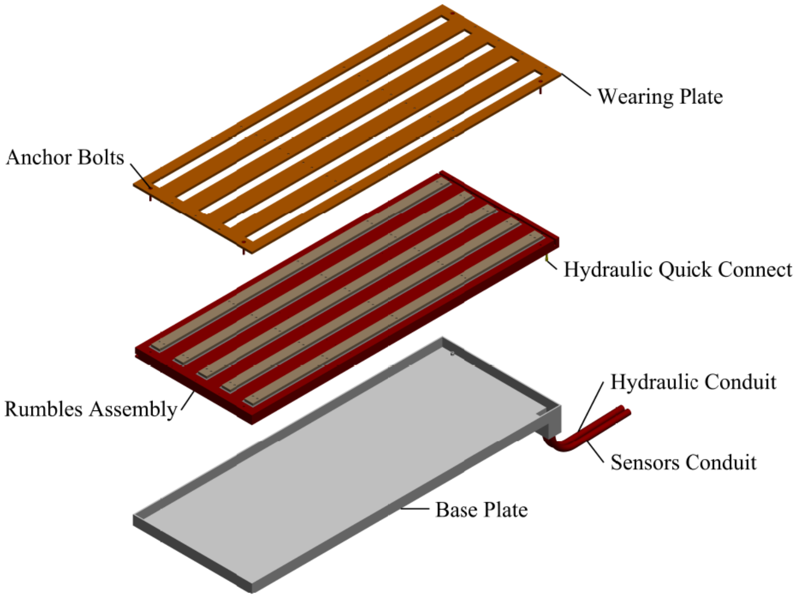


Figure 7. Expanded Isometric View

Key components of the Rumbles Assembly illustrated in Figure 8 include:

1) A single Base Plate designed so an array of rumbles can be assembled. To make an array, the rumble Assembly Frame is slid together, glued and locking pins placed.
2) A Hydraulic Lifting Tube with the Hydraulic Base and Hydraulic Cover protects the tube.
3) A Rubber Seal that prevents water and dirt from entering.

General Operations Description**:**

Initially, the Rumble Plates are flush with the Wearing Plate and roadway. To lower the unit, fluid is removed from the Hydraulic Lifting Tube. The resulting vacuum created, along with the Rubber Seal push down, causes a Locking Plate and the Rumble Plate to lower. To return to the initial position, fluid is pumped into the Hydraulic Lifting Tube, causing the Locking Plate and Rumble Plate to rise back into the initial position, which is flush with the top of the Wearing Plate.

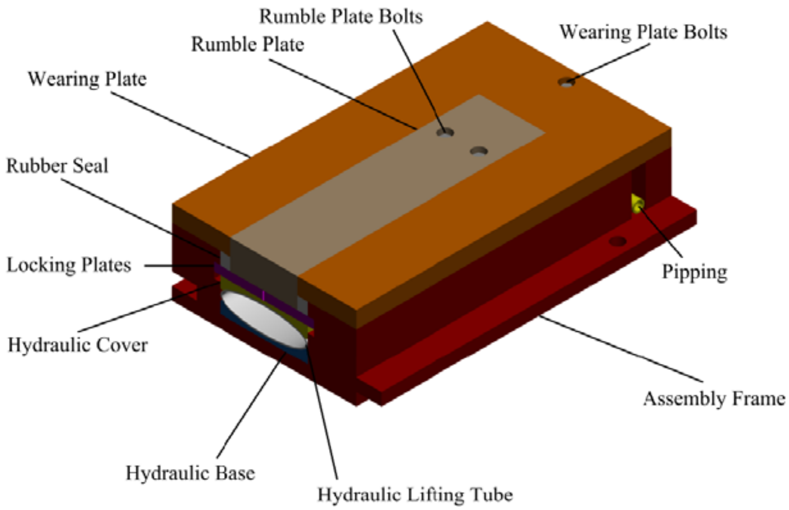


Figure 8. Cross-Sectional View of the Rumbles Assembly and Wearing Plate for a Single Rumble

Third Design and Prototype

Figure 9 illustrates the third alternative for the implementation of the DRTRS. The rumble effect is created by a set of rumble units illustrated in Figure 10. These units include three hydraulic actuators to lower or raise C Channel beams. Figure 11 presents a zoomed view of the most important components of the rumble strip units, including a hydraulic actuator, the C Channel beam, a support column, and a base plate. The rumble units are separated by spacer sections, made of structural steel angles, and filled with concrete to provide stiffness and stability. Rumble strips and spacer sections are bolted to a steel box frame, which will be attached to the road using studs and epoxy. Rumble strips and spacer units can be disassembled separately without the need to remove the box frame. Hydraulic lines connect the actuator to a hydraulic pump and control unit, which will be placed in an appropriate box on the side of the road. Appendix B provides a detailed structural analysis of this design.

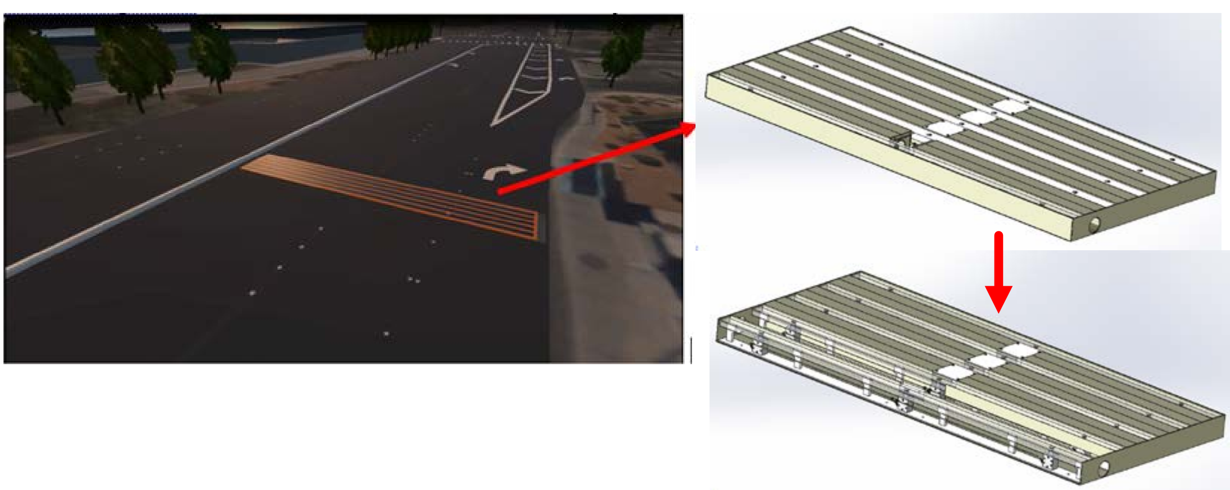


Figure 9. Schematic of the DRTRS Third Design in the Field

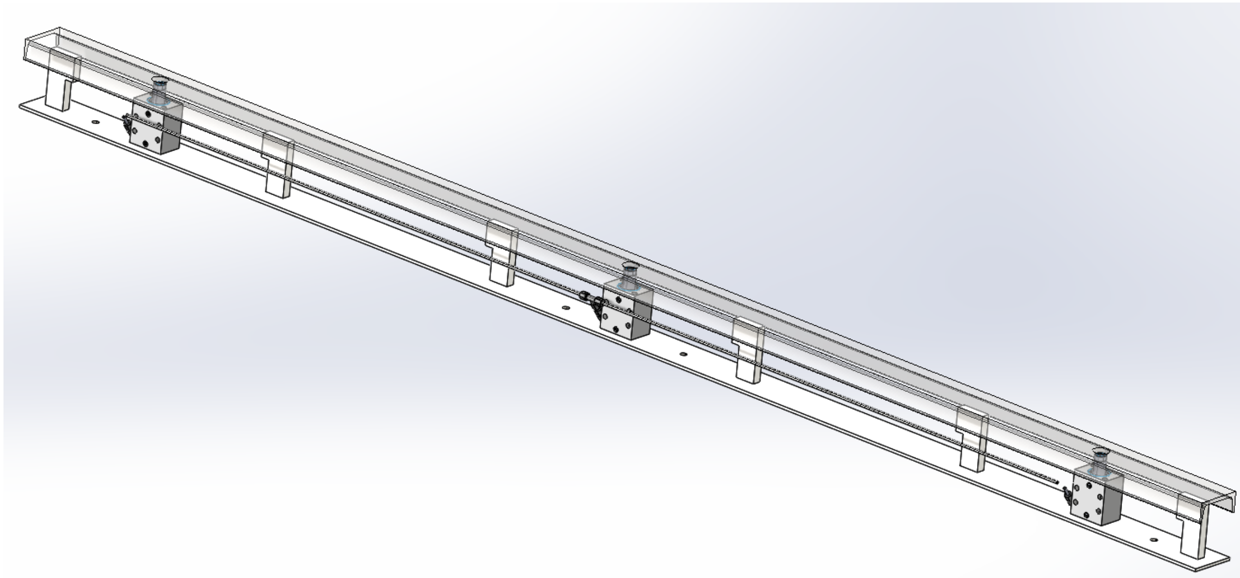


Figure 10. A Unit Box Beam and Roller Assembly of the DRTRS Third Design

The default position of the rumble strips is to have the upper surface of the C Channels flush with the road, spacer units, and upper edges of the box frame. The rumble strips can be activated by either pedestrian push buttons, traffic signal controllers, signals from the buses, and/or vehicle/pedestrian detection systems. When a signal is sent to the system, the hydraulic actuators will lower the rumble strips' C Channels; the resulting recesses create the transverse rumble strip effect.

The proposed modular design is compact, less than 6-inches deep, allowing placement within asphalt without the need for added support or preparation. The rumble units are four inches wide. They are spaced eight inches from each other and are transverse to the flow of vehicular traffic. The C Channel beams create a rumble of 0.5 inches deep. These dimensions were chosen based on the existing literature about conventional transverse rumble strips. Although different jurisdictions use different dimensions, the ones chosen for the DRTRS are consistent with most jurisdictions. However, the spacing and depth of the DRTRS can be changed relatively easily. The design is robust with relatively few components. It was developed to maintain functionality under various conditions including severe temperature variations, as well as rain, snow, and dirt. The use of hydraulic power enhances safety because no electric lines will be used. The DRTRS can be installed and uninstalled quickly. The DRTRS can either be installed directly into the roadway or into an assembly base. The assembly base enables the quick removal and replacement of the DRTRS. This allows minimum maintenance time in the roadway, removal of the unit for pavement operations, and the ability to upgrade roadway sensors, which can be installed in DRTRS for other traffic management purposes. Support columns are added to carry the load caused by the tires of the passing vehicles when the strips are at the recessed position.

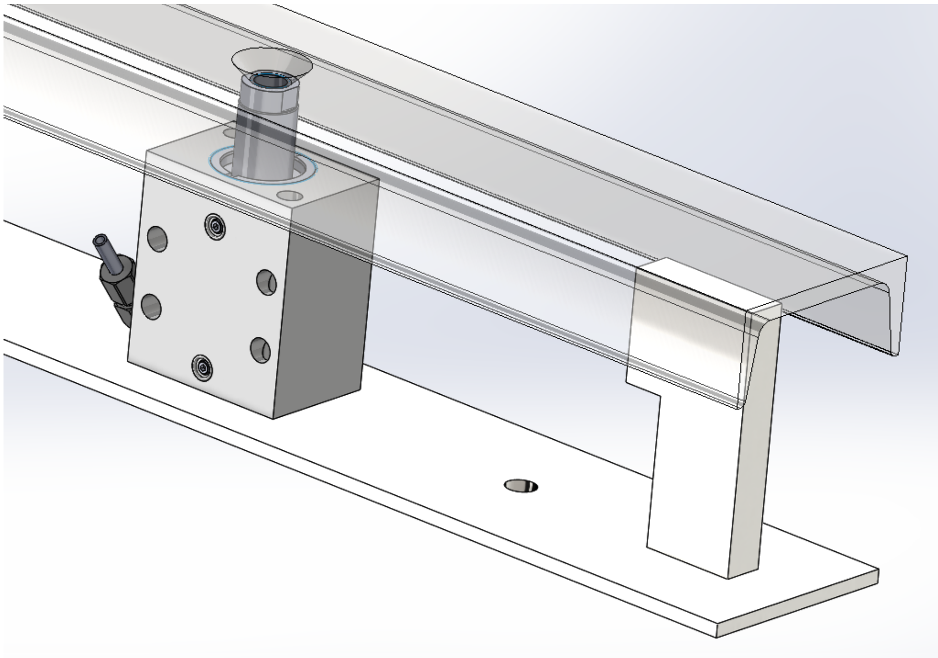


Figure 11. A Zoomed View Showing One Hydraulic Unit and Support of the Demand-Responsive Traverse Rumble Strips

Key components of the DRTRS include:

5) C-channel beams.
6) Hydraulic actuators that lower the C Channel beams to create the rumble effect. These actuators are spring-loaded, which means that only hydraulic power is used to lower the strips.
7) A controller unit that allows various modes of input to the rumble strips.

General Operations Description

At the default position, the C Channels of the rumble strip units are flush with the top of the steel and concrete box frame and roadway. To lower the rumble units, the hydraulic actuators retract, lowering the C Channel beams. To return to the default position, the hydraulic pressure is released, allowing springs within the hydraulic actuators to push the C Channel beams up, causing them to become flush with the top of the box and roadway. The DRTRS can be activated through communication with pedestrians, traffic signal controllers, detection systems, and/or signals from the buses.

TESTING AND RESULTS

Prototyping and Testing of the First Design

Figure 12 shows a prototype of a Unit Box Beam and Roller Assembly and cabinet, which houses all the control hardware and software required for operation. Appendix A provides a detailed design of the reinforced concrete box that houses the Unit Box Beams and Roller Assemblies.

Figure 12. Prototype for the Unit Box Beam and Roller Assembly and Control Cabinet in the Machine Shop

Figure13 illustrates installation in the field of the steel housing for the Unit Box Beam and Roller Assembly.

Figure 13. Installation in the Field of the Steel Housing for the Unit Box Beam and Roller Assembly.

Result and Discussion from the First Design

Appendix E provides a detailed field test methodology for the testing of the first design. Noise and vibration data have been collected using multiple smartphones from field testing, for in-vehicle and on the DRTRS, following a methodology from the literature [12, 13, 14, 15]. For in-vehicle experiments, the x axis served as the lateral axis, z axis as the longitudinal axis, and y axis as the vertical axis. For the case of DRTRS data, the x axis was aligned along the lateral axis of the vehicle, y axis along the longitudinal, and z axis along the vertical axis.

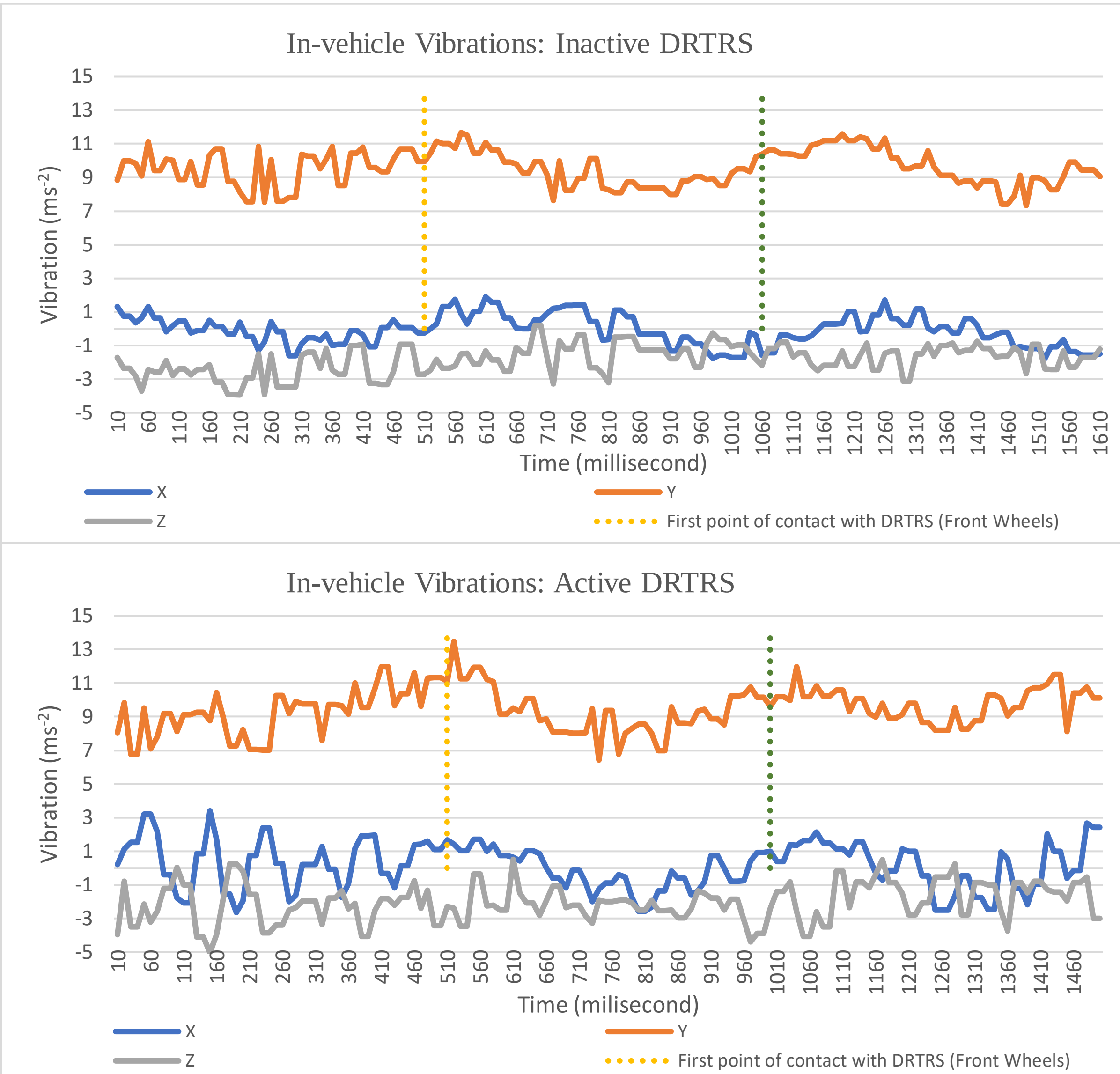


Figure 14: In-vehicle Vibration from a Single Run

According to the device setup, the conventional y axis served as the vertical axis for the cases of inside vehicle data. When the DRTRS was active, there was a consistent peak and trough for the vertical vibrations (y axis); there was higher magnitudes of vertical vibrations when front tires of the truck hit the rumbles, but for the case of rear tires, the magnitude was medium. When the

DRTRS was inactive, there was less in-vehicle vibrations compared to that of active DRTRS. Consequently, active DRTRS yielded vibrations from 13.5 ms^{-2} to 6.5 ms^{-2} which was a feelings of 7 ms^{-2} for in-vehicle drivers to give them a prerequisite alarm for the upcoming pedestrians on the crossings. Inactive DRTRS gave around 3-4 ms^{-2} of in-vehicle vibrations which is much moderate than that of the active ones.

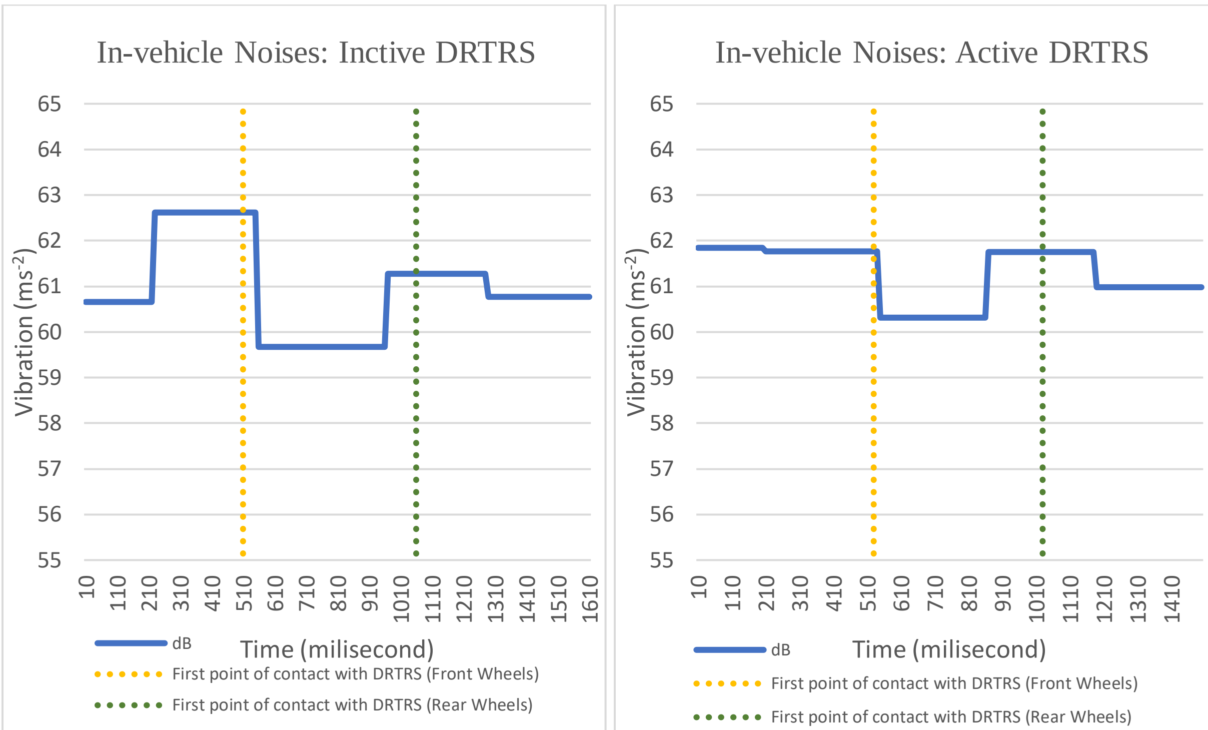


Figure 15: In-vehicle Noises from a Single Run

In-vehicle noise is around 62 dB with the active DRTRS which is around 2 dB higher than the inacitve DRTRS. The first prototype was designed with a single rumble which necessarily made a little extra noise, whereas five sets of continuous rumbles in a DRTRS would yield more noise during its active stage.

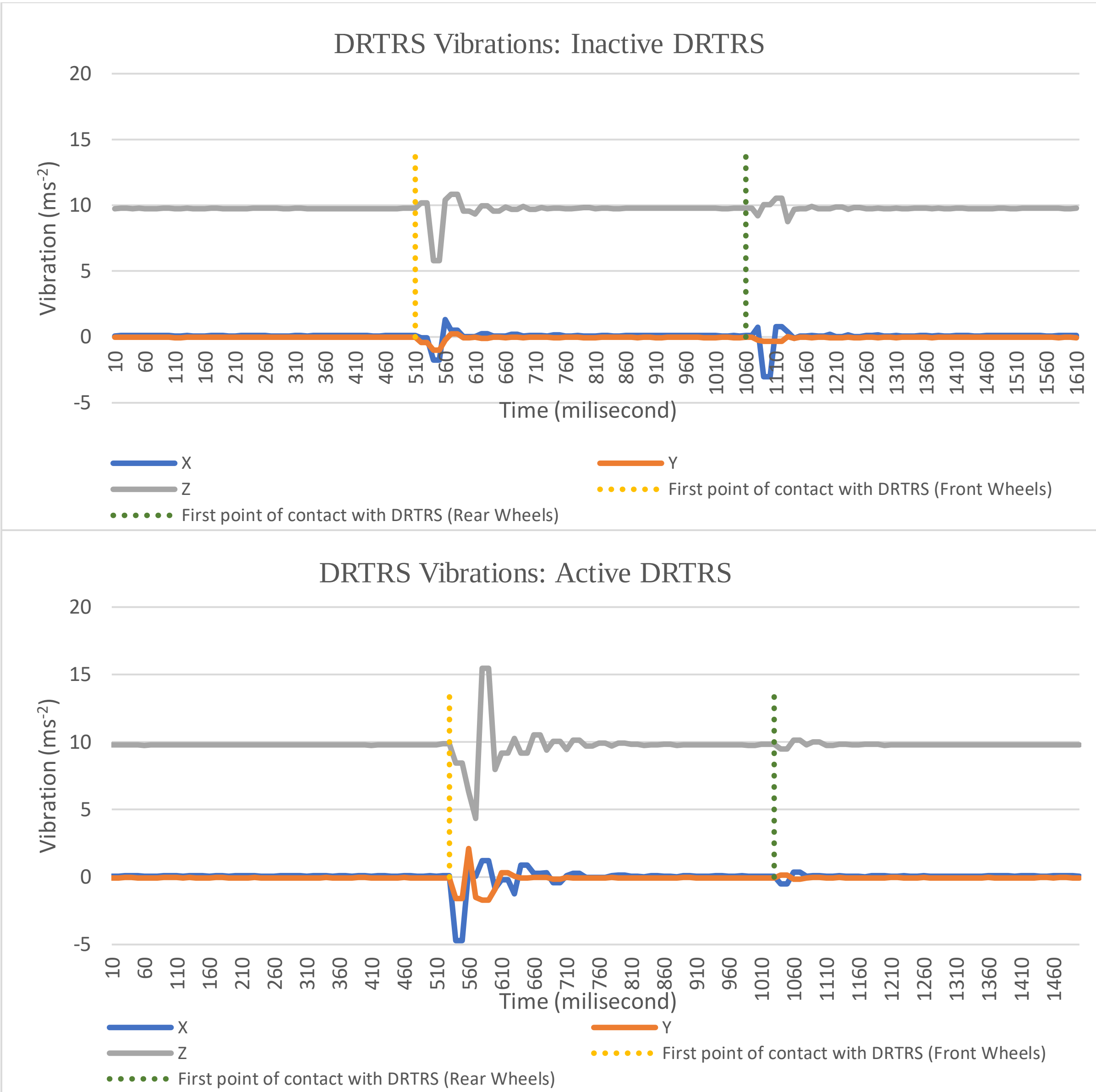


Figure 16: DRTRS Vibration from a Single Run

Because of the balance of the c section beam of the rumble with the roadway surface, when the truck run over the DRTRS, the tires had a good contact with the rumbles that made extra vibrations into the directions of the z axis. Conversely, when the DRTRS was active, no notable vibration was observed as the big tires ran over the niche meagerly touching the c section beam of the rumble. Larger dimension wheels run over the DRTRS with minimum vibration because of their large diameters.

As the testing site of the first design was not long enough to speed up and maneuver, the test runs were performed at a maximum speed of 20 mph. In addition, frequent acceleration and braking to get the truck stopped within the site without hitting other property might have created issues with the unwanted vibrations along the x, y and z axes. Consequently, an elongated continuous road section will yield a better result for the in-vehicle noise and vibration.

The first design of the prototype was deployed with the main concern of the noise and vibrations during the inactive and active stages of the DRTRS. The experiment was performed with the truck running around at 20 mph. In-vehicle noise and vibrations, DRTRS self-vibrations and noise have been recorded for every single run. Paired sample t-tests were performed for in-vehicle vibration to check how it differs when the DRTRS is active or inactive. The hypothesis assumed that there were no significant differences in in-vehicle vibration for active and inactive DRTRS.

Table 1: Paired t-test between In-vehicle Vertical Vibrations

	Mean	t	df	Sig. (2-tailed)	Reject H_0 at 95% C.I.
In-vehicle vertical vibrations during active DRTRS - In-vehicle vertical vibrations during inactive DRTRS	1.57394	3.412	9	.008	Yes

For in-vehicle vertical vibration, the positive t value implies that the hypothesized vertical vibration during the active DRTRS stage is higher than that of the inactive stage, and that is significant at a 95% confidence interval. The half inch depth of the c section bar within the niche during the active stage exhibited substantial vertical vibrations compared to the inactive stage of the DRTRS which is significant to alert drivers.

Site Selection for Testing the Third Design

A prototype for the third design will be installed and tested on East Harmon Avenue within the UNLV jurisdiction. Vehicle speeds varies across the corridor as illustrated by the following analysis. The test corridor was divided into 5 Subsections as depicted in Figure 18. The red circles (**O**) in Figure 18 indicate the positions of the radar guns in each subsection. Ninety-percentile speeds have been considered as the expected analysis speed.

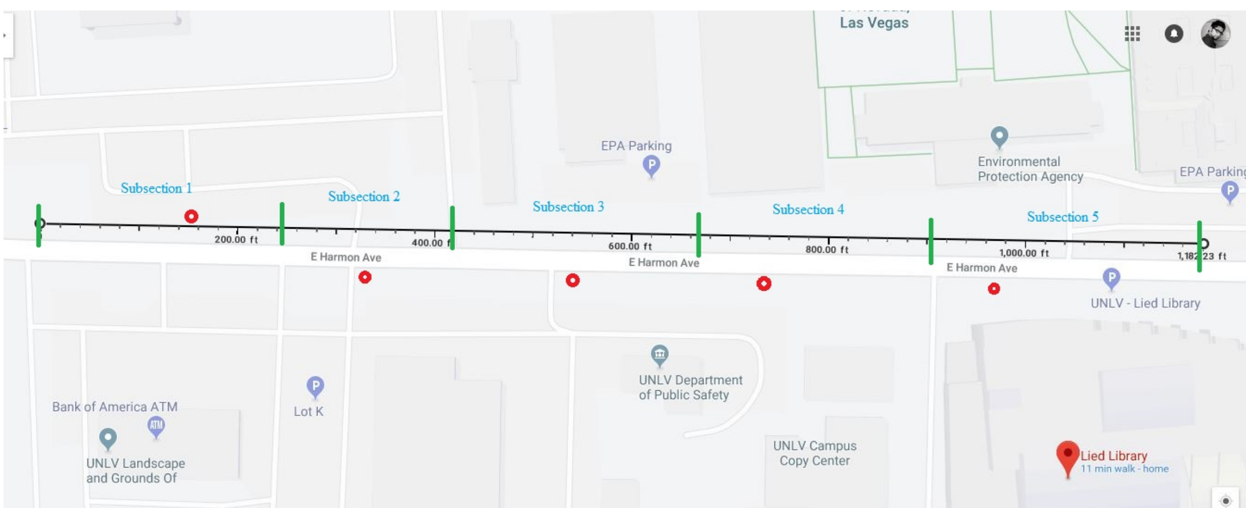


Figure 17: East Harmon Avenue in Front of the Library Divided into Five Subsections.

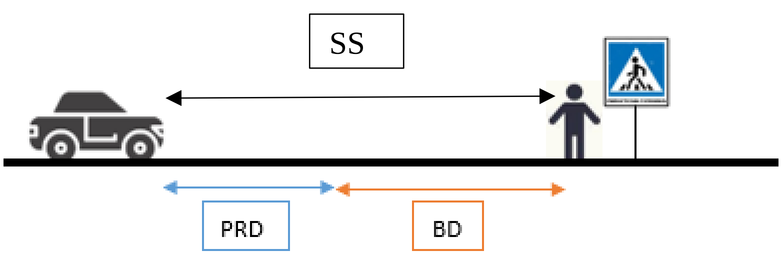
Table 2: Speed Profile on Different Subsections

	Eastbound (mph)	Westbound (mph)
90 percentile speed on subsection 1	23	21
90 percentile speed on subsection 2	26	25
90 percentile speed on subsection 3	26	25
90 percentile speed on subsection 4	26	27
90 percentile speed on subsection 5	18	19

Figure 18: Taking Speed Data at one of the Subsections of East Harmon Avenue

Source: Field data, 2018

The westbound direction of subsection 4 has the highest 90 percentile speed of 27 mph. The sight distance is the length of the road seen by a driver at any time. This distance of visibility must be such that, when a driver is moving on the road, he/she must have time to perform the necessary avoidance maneuvers without colliding with an object. Considering this sight distance, the calculation of the stopping sight distance of the vehicle is divided into two parts: the perception and reaction distance, and the braking distance [16].



Stopping Site Distance (SSD) = Perception reaction distance (PRD) + Braking Distance (BD)

$$SSD = 1.47\,Vt + \frac{V^2}{30((\frac{a}{g}) \pm G)}$$

Here,	Unit
v = Speed when brakes applied	mile per hour *(mph)*
t = Perception reaction time = 2.5s	Second *(s)*
a = Vehicle acceleration = 11.2 ft/s^2	Feet per square second *(ft/s^2)*
g = Acceleration of gravity = 32.2 ft/s^2	Feet per square second *(ft/s^2)*
G = Grading = 0 as the road is flat	No Unit

AASHTO (2001)

Taking 30 mph as the expected or 90-percentile speed, the stopping site distance would be-

$$SSD = 1.47\,Vt + \frac{V^2}{30((\frac{a}{g}) \pm G)}$$

$$SSD = 1.47 \times 30 \times 2.5 + \frac{30^2}{30((\frac{11.2}{32.2}) \pm 0)}$$

$$= 110.25 + 86.25$$

$$= 196.5\ ft.$$

Here,

v = 30 mph

t = Perception reaction time = 2.5s

a = Vehicle acceleration = 11.2 ft/s^2

g = Acceleration of gravity = 32.2 ft/s^2

G = Grading = 0 as the road is flat

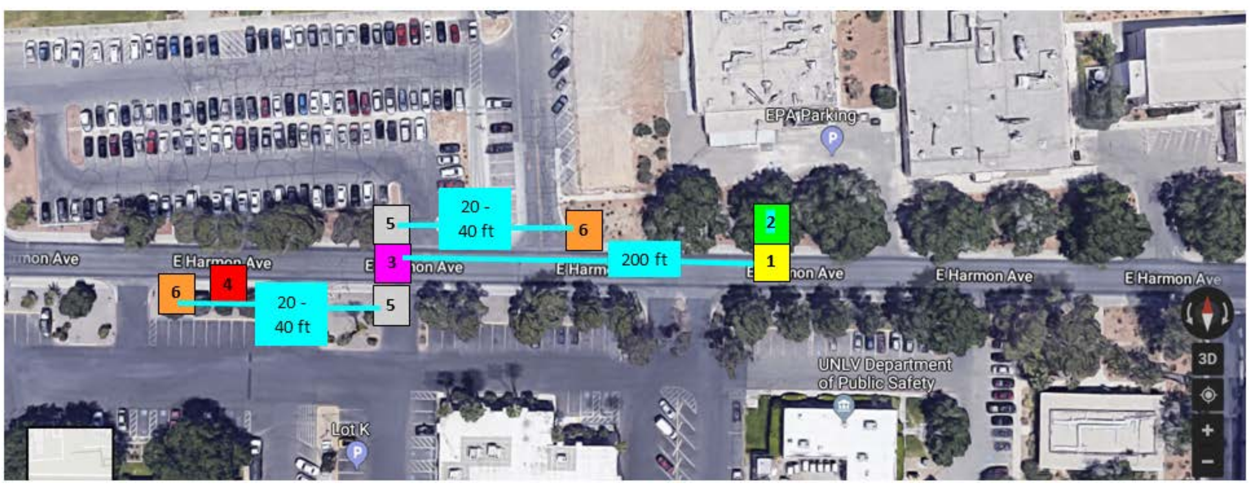


Figure 19: Installation Schematics of the DRTRS, where, (1) Rumble device, (2) Control Cabinet, (3) Crosswalk, (4) Camera, (5) Poles with Push Button, (6) Pedestrian Crossing Signs.

The Distance between the center of the crosswalk (3) and the center of the rumble device (1) is 196.5 feet, and the design distance is 200 ft. [17]. Likewise, the distance between the center of the crosswalk (3) and the pedestrian crossing signs (6) should be between 20 to 40 ft. [18].

Prototyping and Testing of the Third Design

Appendix C provides a detailed Work Plan for the testing of the third design. By the time this report is due, we have completed building this prototype but have not begun testing. A future report submitted to the University Transportation Center administrators will include results from this test.

Similarly, Appendix D provides a survey questionnaire that will be used to collect data about peoples' opinions and attitudes towards the DRTRS traffic safety device. Results from this survey and corresponding analysis will be included in a future report.

POTENTIAL PAYOFF FOR PRACTICE

Alternative Solutions

Other alternative engineering mechanisms, such as rapid-flashing beacons are unlikely to provide the same strong effects as the rumble strips, which is evidenced by fatalities that have occurred in areas with these beacons. One advantage of the proposed DRTRS is the level of rapid vibration/discomfort they produce. Hence, drivers are instinctively and immediately forced to regain their attention to the roadway, even before they see a pedestrian. This is the same proven effect as static transverse or shoulder rumble strips have on drivers. There is no similar product to DRTRS in the market. Available alternatives require drivers to look at the roadway and surrounding infrastructure. Distractions such as cell phone use, impaired driving, interaction with passengers, or external disturbances are frequent and preclude the intended effect of warning drivers about the presence of pedestrians on the roadway. The DRTRS creates the required effect even with driving distractions, as the vibration creates the involuntary reaction of regaining roadway attention. In addition, the audible sound warns pedestrians of the presence of a vehicle. Permanent or static rumble strips lose their effectiveness over time because drivers get used to or even try to avoid them [1] creating additional risks. By regaining the driver's attention, we believe that existing safety improvements, such as rail road crossing arms and flashing beacons can be more effective.

Additional Applications of the Proposed Solution

In addition to pedestrian areas and intersections, there are several other applications of the DRTRS, including railroad crossings, tollbooths, and speed control zones, such as school zones. In the case of school zones, given that the DRTRS is only active during daytime hours, they can be placed in residential neighborhoods, whereas permanent rumbles cannot, due to nighttime nose. Recently, several accidents were attributed to inattentive drivers at these types of locations. The Amtrak accident on US 95A in Nevada where six people were killed is just one example[4]. As already mentioned, rapid-flashing beacons along with railroad crossing arms proved ineffective at regaining the drivers' attention. On average, these devices cost well over $600,000 to install at each crossing. It is anticipated that our DRTRS, in addition to these devices, will be effective at regaining distracted drivers' attention for only an additional fraction of the overall cost of these standard installations. In terms of financial setback to the community, a fatality is estimated to cost more than $5.5M. Similarly, the cost of injuries is extremely high, according to the Highway Safety Manual. On average, a single injury is likely to be more expensive than the cost of the proposed DRTRS.

TRANSFER TO PRACTICE

We are working with the Nevada Division Office of the Federal Highway Administration (FHWA) through an existing Stewardship & Oversight Agreement with NDOT. We have obtained the

[4] http://sanfrancisco.cbslocal.com/2011/06/25/feds-probe-truck-driver-killed-amtrak-crash/

required permission for the proposed on-the-road field testing and to move the technology forward after the objectives of this project are completed.

The UNLV Office of Economic Development (OED), as the designated intellectual property management organization of UNLV, will primarily lead efforts to achieve successful commercialization of intellectual property from this project. The primary commercialization strategy for the DRTRS technology will be to seek partnerships to expedite technology development, followed by the licensing of intellectual property resulting from the project. Specifically, the OED is working with the lead investigator to establish industry partnerships, public sector partnerships, research collaborations, and the licensing of intellectual property to a commercial partner, ideally with a Nevada presence, capable of fully exploiting the technology in the marketplace.

We are currently working closely with NDOT and the Nevada Governor's Office of Economic Development (GOED), so that after successful testing, the DRTRS is adopted as a standard safety device. This is expected to translate into substantial traffic safety benefits, which include saving lives and reducing injuries and property damage, as well as other negative externalities associated with crashes, such as non-recurrent congestion and emissions.

In addition to the expected traffic safety benefits, this project includes a goal of enhancing economic growth within the State of Nevada. The project should result in the following returns on investment:

1. <u>Intellectual Property and Brand Value.</u> The proposed project will result in the creation, identification, and protection of new intellectual property in the form of patents and copyrights, visible participation in transportation safety projects with both regional and global applications, and recognition for Nevada as a leading innovator in the field of mobility and transportation solutions. Note that UNLV OED has filed a U.S. utility patent application [19] covering the DRTRS technology in anticipation of receiving adequate project funding to continue development and commercialization of the technology.
2. <u>Technology Transfer.</u> The licensing of project intellectual property will result in IP revenues, which over time, will provide a return to help sustain DRTRS technology research and innovation. The OED will seek licensees and partnerships with relevant companies and industries that might benefit from DRTRS technology.
3. <u>Start-up Acceleration & Industry Development.</u> Using lean start-up methodology, coupled with resources from the Nevada Small Business Development Center, the UNLV Center for Entrepreneurship, and students from the UNLV Lee School of Business, the OED will work to identify specific market applications as well as the suitability of the DRTRS technology as the basis of a Nevada-based start-up company. Furthermore, ongoing translational research, prototype development, field testing, and ultimately technology commercialization itself will be the significant direct drivers of regional economic and workforce development. We anticipate indirect impacts to the region as new DRTRS related products and services will be developed and commercialized by both existing and new companies to fully exploit the technology.

CONCLUSIONS AND NEXT STEPS

This study designed and evaluated three alternative mechanisms for the deployment of Demand Responsive Transverse Rumble Strips (DRTRS). A first prototype using an electric actuator was tested on a private facility at the University of Nevada Las Vegas. The objective of the test was to evaluate the level of vibration and noise generated by the DRTRS as well as durability and reliability. Results illustrate the vibration and noise generated by the prototype. Our evaluation of the three alternative mechanism concluded that an innovative hydraulically-activated design is the best approach to deploy the DRTRS. The proposed DRTRS apparatus is modular, and the mechanical components of the DRTRS units are reliable with few components. The hydraulic system is expected to require some maintenance. However, this system is placed in a cabinet outside of the travel lanes. The DRTRS deployment cost is comparable to existing solutions for intersections, school zones, toll lanes, and speed control zones. A second prototype based on the hydraulic system was built and it is about to be tested on a public facility at the University of Nevada Las Vegas. Results from this testing will provide information about its effectiveness and potential insights to further improve our design to make it even more cost and safety effective. A future report will include these results and the corresponding analysis.

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APPENDIX A

Detailed design of the reinforced concrete box that houses the Unit Box Beams and Roller Assemblies for the first Design of the DRTRS

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GENERAL NOTES

GENERAL

THESE NOTES ARE GENERAL AND APPLY TO THE ENTIRE PROJECT EXCEPT WHERE SPECIFICALLY INDICATED OTHERWISE.

STRUCTURAL DIMENSIONS CONTROLLED BY OR RELATED TO MECHANICAL OR ELECTRICAL EQUIPMENT SHALL BE COORDINATED BY THE CONTRACTOR PRIOR TO CONSTRUCTION. BOLT SIZES, TYPES, AND PATTERNS SHALL BE VERIFIED WITH THE MANUFACTURER. ALL BOLT PATTERNS SHALL BE TEMPLATED TO INSURE ACCURACY OF PLACEMENT.

MECHANICAL AND ELECTRICAL EQUIPMENT SUPPORTS, ANCHORAGES, OPENINGS, RECESSES AND REVEALS NOT SHOWN ON THE STRUCTURAL DRAWINGS BUT REQUIRED BY OTHER CONTRACT DRAWINGS, SHALL BE PROVIDED FOR PRIOR TO PLACING CONCRETE.

STRUCTURAL DRAWINGS SHALL BE USED IN COORDINATION WITH MECHANICAL, ELECTRICAL, ARCHITECTURAL, CIVIL DRAWINGS AND SHOP DRAWINGS PROVIDED BY MANUFACTURERS OF EQUIPMENT.

STRUCTURES HAVE BEEN DESIGNED FOR CRANING, OPERATIONAL, TRAFFIC, AND BACKFILL LOADS ON THE COMPLETED STRUCTURES. THE STRUCTURES HAVE NOT BEEN DESIGNED TO RESIST THESE LOADS WHILE ONLY PARTIALLY CONSTRUCTED. DURING CONSTRUCTION, THE STRUCTURES SHALL BE PROTECTED FROM ALL CONSTRUCTION LOADS BY BRACING AND BALANCING UNTIL ALL STRUCTURAL ELEMENTS ARE IN PLACE, AND ALL CONCRETE HAS REACHED THE SPECIFIED 28 DAY COMPRESSIVE STRENGTH. OVERLOADING OF ANY STRUCTURAL ELEMENT IS PROHIBITED.

UNLESS OTHERWISE SHOWN, ON ALL STRUCTURAL DRAWINGS THE FINISHED GRADE AROUND STRUCTURES IS SHOWN THUS, INDICATING EITHER GROUND SURFACE, TOP OF CONCRETE SLAB, OR AC PAVEMENT. FOR DETAILS OF FINISH SURFACES SEE CIVIL AND ARCHITECTURAL DRAWINGS.

CONCRETE

UNLESS OTHERWISE NOTED OR SPECIFIED, ALL STRUCTURAL CONCRETE SHALL BE A CLARK COUNTY PUBLIC WORKS QUALIFIED CLASS E MIX AND DEVELOP A MINIMUM COMPRESSIVE STRENGTH OF 4500 PSI IN 28 DAYS.

REINFORCEMENT STEEL SHALL BE DEFORMED BARS CONFORMING IN QUALITY TO THE REQUIREMENTS OF ASTM A-615, "SPECIFICATIONS FOR DEFORMED BILLET-STEEL BARS FOR CONCRETE REINFORCEMENT", GRADE 60

ALL DETAILING, FABRICATION AND PLACING OF REINFORCING BARS, UNLESS OTHERWISE INDICATED, SHALL BE IN ACCORDANCE WITH ACI-315, "MANUAL OF STANDARD PRACTICE FOR DETAILING REINFORCED CONCRETE STRUCTURES", LATEST EDITION.

TOLERANCES IN PLACING REINFORCEMENT SHALL BE:

+/- 3/8 INCH FOR MEMBERS WITH DEPTH D </= 8 INCHES
+/- 1/2 INCH FOR MEMBERS WITH DEPTH D > 8 INCHES

DOWELS, PIPE, CONDUIT AND OTHER INSTALLED MATERIALS AND ACCESSORIES SHALL BE HELD SECURELY IN POSITION WHILE CONCRETE IS BEING PLACED.

ALL GROUT SHALL BE NON-SHRINK GROUT, UNLESS INDICATED OTHERWISE.

METAL CLIPS OR SUPPORTS SHALL NOT BE PLACED IN CONTACT WITH THE FORMS OR THE SUBGRADE. CONCRETE BLOCKS (OR DOBIES) SUPPORTING BARS ON SUBGRADE SHALL BE IN SUFFICIENT NUMBERS TO SUPPORT THE BARS WITHOUT SETTLEMENT, BUT IN NO CASE SHALL SUCH SUPPORT BE CONTINUOUS.

UNLESS OTHERWISE INDICATED ON THE DRAWINGS, LAPS OF REINFORCEMENT SHALL BE AS SHOWN ON DETAIL S-143.

REINFORCING BARS AND ACCESSORIES SHALL NOT BE IN CONTACT WITH ANY PIPE, PIPE FLANGE OR METAL PARTS EMBEDDED IN CONCRETE, A MINIMUM OF 2 INCHES CLEARANCE SHALL BE PROVIDED AT ALL TIMES.

ALL ITEMS EMBEDDED IN CONCRETE SHALL BE SPACED ON CENTER AT LEAST 4 TIMES THEIR OUTSIDE DIMENSION. THE OUTSIDE DIMENSION SHALL NOT EXCEED ONE THIRD OF THE MEMBER THICKNESS

ELECTRICAL CONDUIT EMBEDDED IN CONCRETE SHALL NOT BE SPACED CLOSER THAN 3 OUTSIDE DIAMETERS ON CENTER.

UNLESS OTHERWISE SHOWN ON THE DRAWINGS CONCRETE COVER FOR REINFORCING BARS SHALL BE AS FOLLOWS:

FOR SURFACES IN CONTACT WITH WATER OR WEATHER AND FORMED SURFACES IN CONTACT WITH EARTH ———— 2"

FOR CONCRETE NOT EXPOSED TO WEATHER, OR IN CONTACT WITH WATER OR EARTH ———— 1 1/2"

STRUCTURAL STANDARD DETAILS

DETAILS ON GS SHEETS ARE PART OF STANTEC'S STRUCTURAL STANDARD DETAILS.

THESE DETAILS ARE TO BE USED WHEN REFERRED TO OR WHEN NO OTHER MORE RESTRICTIVE OR DIFFERENT DETAILS ARE INDICATED ON THE DRAWINGS.

STRUCTURAL STEEL

STEEL CONSTRUCTION SHALL CONFORM TO THE SPECIFICATIONS AND STANDARDS AS CONTAINED IN THE LATEST EDITION OF THE LRFD MANUAL OF STEEL CONSTRUCTION.

ALL STEEL MEMBERS AND CONNECTORS SHALL BE HOT-DIP GALVANIZED AFTER FABRICATION.

OTHER SHAPES, BARS, PLATES AND SHEETS SHALL BE OF STEEL MEETING ASTM A-36 SPECIFICATIONS.

HSS SHALL BE OF STEEL MEETING ASTM A-500 GRADE B.

ALL WELDING SHALL BE BY THE SHIELDED ARC METHOD AND SHALL CONFORM TO AWS CODE FOR ARC AND GAS WELDING IN BUILDING CONSTRUCTION. QUALIFICATIONS OF WELDERS SHALL BE IN ACCORDANCE WITH THE SPECIFICATIONS FOR STANDARD QUALIFICATION PROCEDURE OF THE AWS.

DESIGN CRITERIA

DESIGN IN ACCORDANCE WITH THE 2012 EDITION OF THE INTERNATIONAL BUILDING CODE AND AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, EXCEPT WHERE OTHER APPLICABLE CODES OR THE FOLLOWING NOTES ARE MORE RESTRICTIVE.

LOADINGS:

SOIL LOADS:

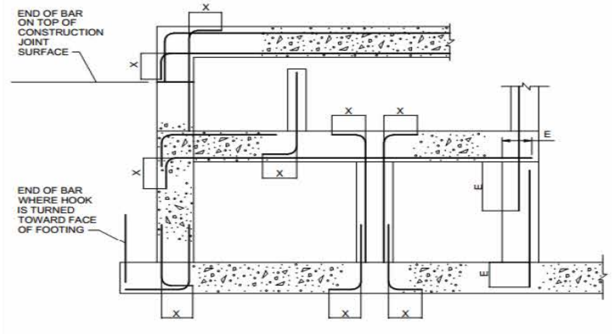
ALLOWABLE BEARING PRESSURE
DEAD PLUS LIVE (PSF): ------------------------------ 3000
ALL LOADS (W/ WIND & SEISMIC) (PSF): ---------------------- 4000

LIVE LOADS (AASHTO HL-93 TRAFFIC):

MAX DESIGN TRUCK AXLE (KIPS): ---------------------------- 32
MAX DESIGN TANDEM AXLE (KIPS): -------------------------- 25
DYNAMIC IMPACT LOAD (% OF LOAD): ------------------------ 33%
BREAKING FORCE (% OF LOAD): ----------------------------- 25%

NDOT NOTES

ALL ROADWAY WORK WITHIN NDOT RIGHT-OF-WAY MUST BE PERFORMED ACCORDING TO LATEST NDOT REQUIREMENTS AND SPECIFICATIONS.

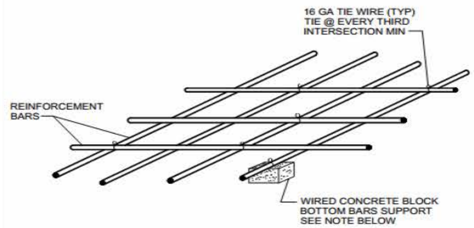


SECTION

LENGTH (*)			
BAR SIZE	HOOK X	LAP	EMBEDMENT E
#3	6"	16" (16")	12" (12")
#4	8"	16" (19")	12" (14")
#5	10"	19" (25")	14" (19")
#6	12"	28" (36")	21" (27")
#7	14"	46" (59")	35" (45")
#8	16"	59" (77")	45" (59")
#9	20"	75" (97")	57" (74")
#10	22"	95" (123")	73" (94")
#11	24"	116" (151")	89" (116")

* USE LENGTH IN PARENTHESIS FOR WALL HORIZONTA REBARS AND SLAB BARS WITH 12" OR MORE OF FRESH CONCRETE UNDERNEATH

STAN
LENG

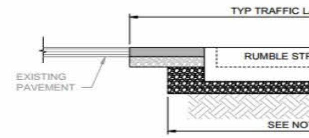


NOTE:

WIRED CONCRETE BLOCK COMPRESSIVE STRENGTH SHALL MATCH OR EXCEED SPECIFIED STRENGTH. METAL BAR SUPPORTS NOT ALLOWED FOR SLAB ON GRADE.

REINFORCEMENT SUPPORT

S-204



NOTES:

1. SLOPE TRENCH SIDES IN ACCORDANCE WITH THE AGENCY REQUIREME AND PROVIDE TRENCH SHORING.
2. NO STONES GREATER THAN 3" PERMITTED IN TRENCH.
3. BOX OUTSIDE DIMENSION PLUS 6" MIN EA SIDE.
4. FOLLOW NDOT STANDARD SPECIFICATIONS FOR AC PAVEMENT SUBGR

REV	DATE	BY	DESCRIPTION

SCALE
AS SHOWN

WARNING
0 ½ 1
IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE

DESIGNED IBB
DRAWN IBB
CHECKED DR

Stantec

UNLV COLLEGE OF ENGINEERING

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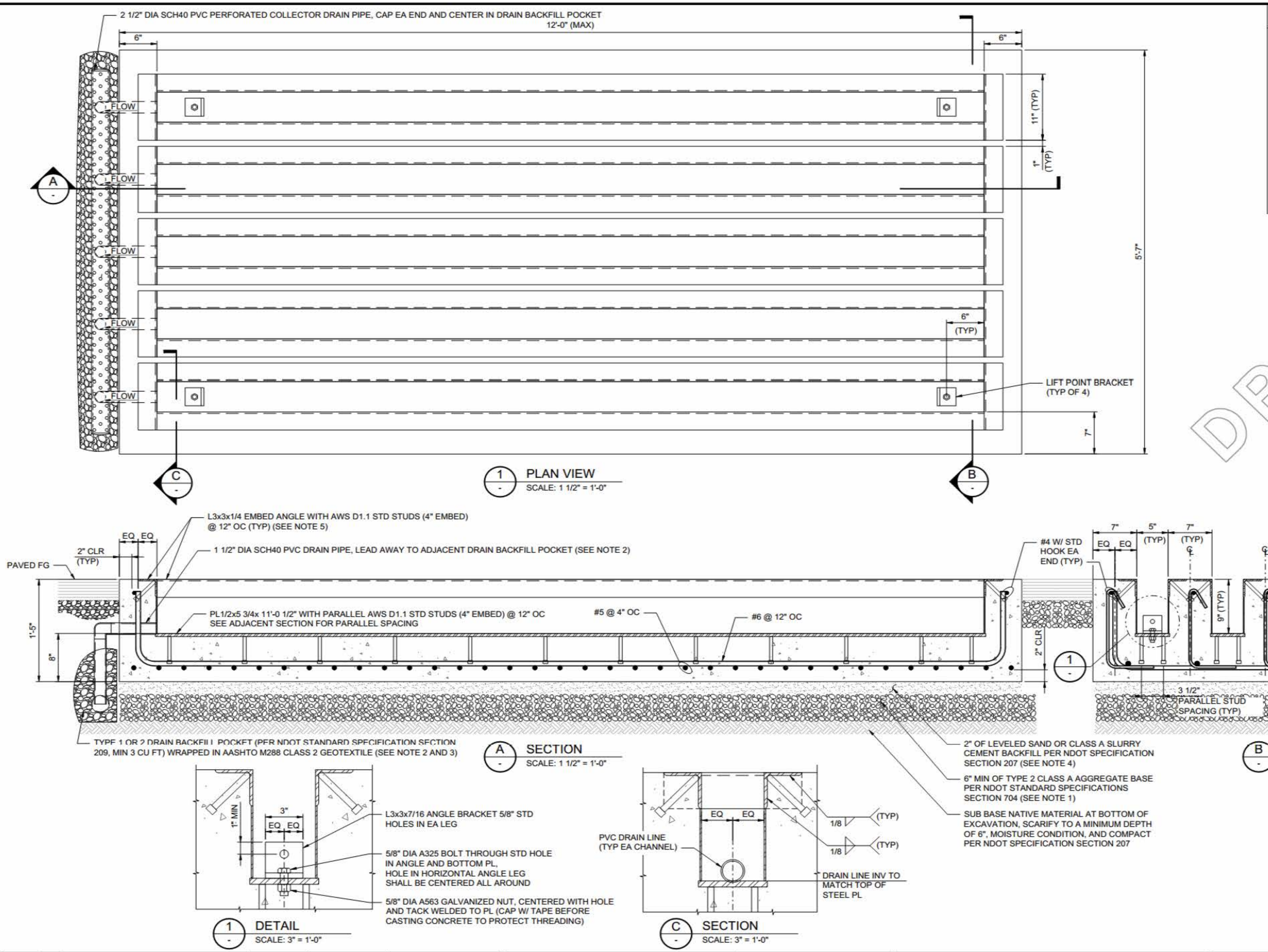
User: ibatilov



REV	DATE	BY	DESCRIPTION	SCALE	WARNING		
				AS SHOWN	0 ½ 1 IF THIS BAR DOES NOT MEASURE 1" THEN DRAWING IS NOT TO SCALE	DESIGNED IBB DRAWN IBB CHECKED DR	Stantec — UNLV COLLEGE OF ENGINEERING

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UNLV

DRT Rumble Strip Box
Structural Analysis & Design

STRUCTURAL CALCULATIONS

Draft

May 14, 2018

Stantec

UNLV College of Engineering
DRT Rumble Strip Box
JOB NO. 181307096

TABLE OF CONTENTS

Stantec
3010 W. Charleston Blvd.
Suite 100
Las Vegas, NV 89102

Calculations by: Iani Batilov, P.E.
Checked by: Douglas Rounds, P.E., S.E.

SECTION 100
GENERAL

STRUCTURAL DESIGN CRITERIA

Client:	UNLV College of Engineering
Project:	DRT Rumble Strip Box

Lead Structural Engineer:	Douglas Rounds	**Date:**	02-13-2018
Project Manager; Office:	Margaret Regan; Las Vegas	**Job Number:**	181307096

This design criterion applies to all structures.

DESIGN CALCULATIONS, METHODS, AND ASSUMPTIONS

Calculations will be done in accordance with the Stantec Best Practices – Structural Calculation Procedures. A title page and table of contents shall be included for each set of calculations greater than five sheets long.

Structures shall be designed in accordance with sound engineering principles based on the references listed below.

DESIGN REFERENCES:

2012 IBC	International Code Committee (ICC) - International Building Code
ASCE 7-10	Minimum Design Loads For Buildings and Other Structures LRFD
AASHTO 2012	Bridge Design Specifications
AISC 360-10	Specification for Structural Steel Buildings Seismic
AISC 341-10	Provisions for Structural Steel Buildings Structural
AWS D1.1-04	Welding Code – Steel
ACI 318-11	Building Code Requirements for Structural Concrete

Client: UNLV College of Engineering

Project: DRT Rumble Strip Box

DESIGN INFORMATION AND

1. The reinforced concrete box houses 5 rumble strip mechanisms. Refer to Dimensions summary sheet.
2. The reinforced concrete box has been analyzed as a spread footing foundation with superimposed HL- 93 traffic loading mimicking governing design truck or design tandem loads.

Client:	UNLV College of Engineering
Project:	DRT Rumble Strip Box

LOADIN

Live loads:	HL-93 Design Truck:	8.0 kip Front Axles 32 kip Back Axles (Max)
	HL-93 Design Tandem:	25 kip axles
	Tire Contact Area	10"x20"

Note:

GEOTECHNICAL

Allowable Bearing Pressure:

	Based on IBC Section 1806 Table 1806.2	3000 psf on Sandy Gravel and/or Gravel
Groundwater Elevation:	Not encountered	
Friction Factor:	**Based on IBC Section 1806 Table 1806.2**	0.35 0.30
Soil Weight:	Structural fill/ native gravels	130 pcf

Client: UNLV College of Engineering

Project: DRT Rumble Strip Box

STRUCTURAL

Concrete: **4500** psi - STRUCTURAL (all structural applications)

Reinforcing: Grade 60 - all applications.

Steel: Wide Flange Shapes - ASTM A992 Angles and C Channels – ASTM A36 Structural Tubing - ASTM A500, Grade B Plates - ASTM A36

S T R U C T U R A L   D E S I G N   C R I T E R I A

Client: UNLV College of Engineering

Project: DRT Rumble Strip Box

WEIGHTS OF

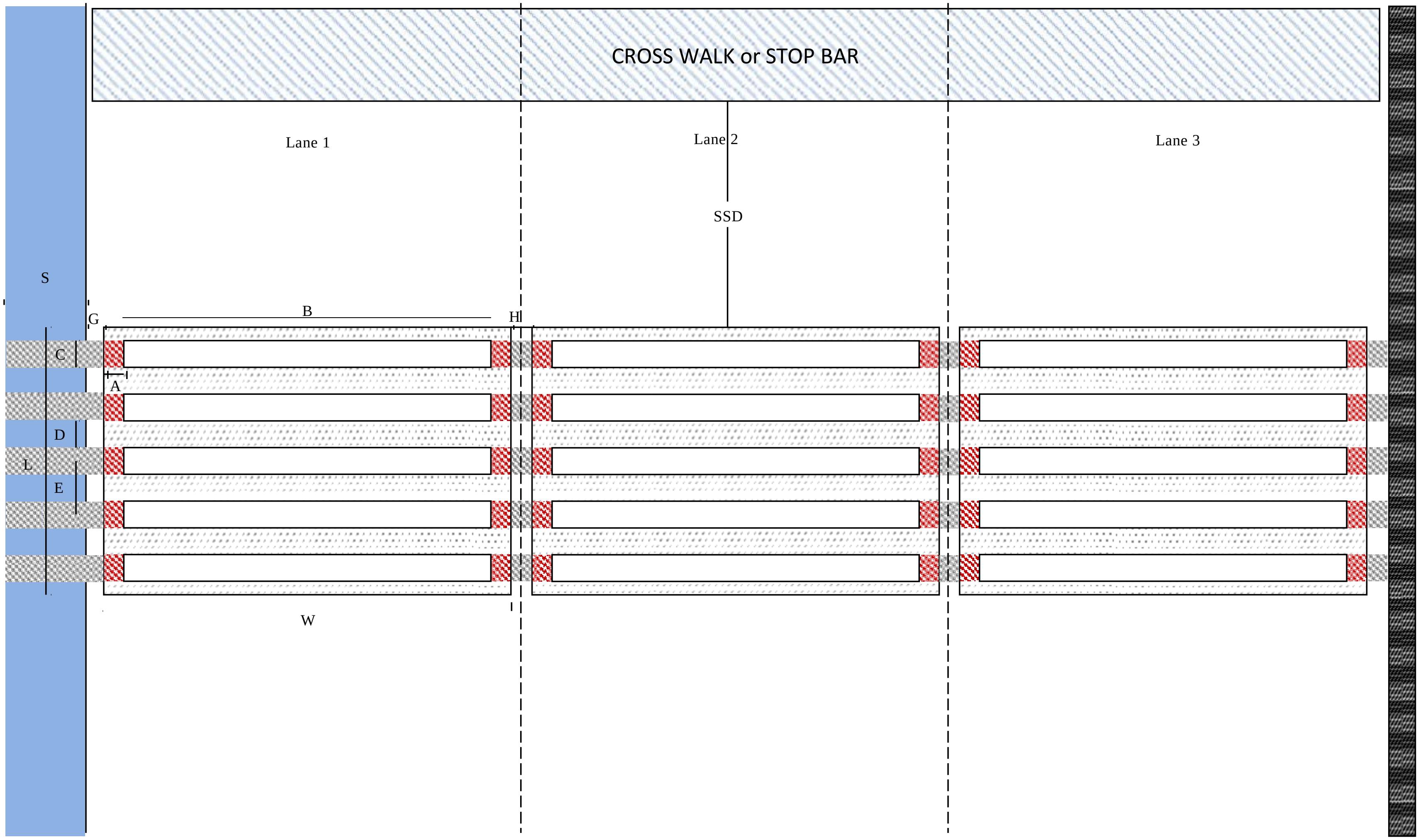
Concrete:	150 pcf
CMU:	125 pcf
Steel:	490 pcf

SAFETY FACTORS

Buoyancy:	NA	
Overturning:	1.50 Static Loads	1.10 Seismic Loads
Sliding:	1.50 Static Loads	1.10 Seismic Loads

OTHER

Demand Responsive Transverse Rumble Strip (DRTRS) – "Sky view"



A= Distance from the edge of the housing box to a rumble

B= Length of a DRTRS => min = 10 ft – 2*A; Max = 12 ft – 2*A

C= Width of a DRTRS = 5 inches

D= Gap between two DRTRS = 7 inches

E= Center to center distance between two DRTRS = 12 inches

G= Distance from the left lane mark to the housing box

H= Distance between two adjacent housing boxes

S= Shoulder with

L= Length of the housing box for a set of DRTRS

W= Width of the housing box for a set of DRTRS

SSD = Stopping sight distance

Permanent rumble strips on the housing box

Permanent rumble strips on the pavement and shoulder

R = Maximum depth of the rumbles = 0.5 inches

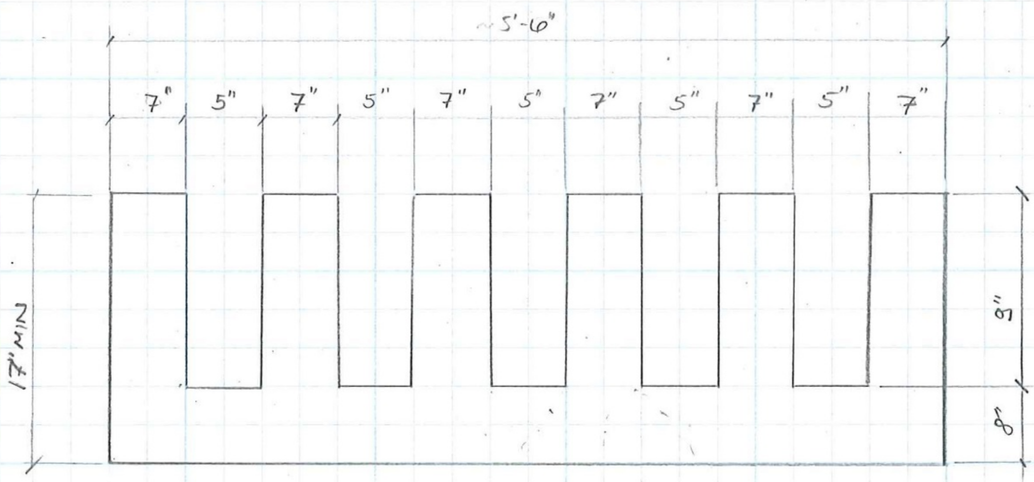
By I. BATILOV Date 1/29/18 Client UNLV Sheet 1 of

Chkd. By D. ROUNDS Description DRT RUMBLE STRIP Job No. 181307096

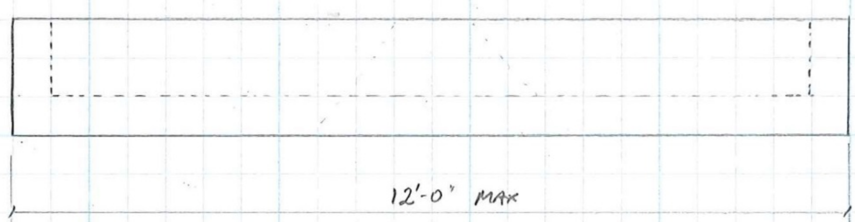
Design Task SUMMARY OF LOADS / DIMENSIONS

REFERENCES

THE DRT RUMBLE STRIP WILL BE INSTALLED LEVEL W/ STREET F.G. AND BEAR ON SUBGRADE. ONE FULL DRT ASSEMBLY WILL HOUSE (5) PNEUMATIC MECHANISMS ~10' LONG × 5" WIDE × 9" DEEP HOUSED IN A STEEL LINED POCKET AND ALL ENCASED IN A CONCRETE BOX. THE BOX NEEDS TO BE DESIGNED FOR HL-93 TRAFFIC LOADING PER AASHTO, AND APPLICABLE SELF-WT IN THE LOAD CASE IT IS CRANE HOISTED AND INSTALLED. CLIENT ALSO REQUESTS THAT RUMBLE STRIP CAN SUPPORT A 125 PSI SUPER AXLE TYPE LOADING



RUMBLE STRIP HOUSING SECTION



TRAFFIC LOADING (LL)

HL-93 — AASHTO LRFD 2012 3.6.1.2

↳ DESIGN TRUCK (32ᴷ AXLE MAX) — 3.6.1.2.1

↳ DESIGN TANDEM (PAIR OF 25ᴷ AXLES 4'-0" APART) — 3.6.1.2.2

TIRE AREA: 10"L × 20"W (SINGLE OR DOUBLE TIRE) — 3.6.1.2.3

5 squares per inch

MWH now part of Stantec

By J. BATILOV Date 1/29/18 Client UNLV Sheet 2 of

Chkd. By Description Job No. 181307096

Design Task

DYNAMIC LOAD ALLOWANCE (IM)	REFERENCES
THE DYNAMIC LOAD IS A FACTOR APPLIED TO THE STATIC DESIGN TRUCK AND DESIGN TANDEM LOADS PART OF VEHICULAR LIVE LOADS (LL).	AASHTO LRFD 3.6.2.1 TABLE 3.6.2.1-1
PER TABLE 3.6.2.1-1, FOR ALL OTHER COMPONENTS BESIDES DECK JOINTS, IM IS 33% SO IM FACTOR IS: $(1+IM/100)$ $= (1+33\%/100)$ $= 1.33$	
DEAD LOAD OF STRUCTURAL COMPONENTS (DC)	
$\gamma_{steel} = 490$ PCF $= 0.284$ lbf/in^3	AISC 360-10 MANUAL TABLE 1-11
THE RUMBLE STRIP BEAM: HSS6x5x1/4, $W_t = 17.32$ PLF	
LEDGE EMBED ANGLES: L3x3x1/4, $W_t = 4.90$ PLF	
BOTTOM OF CHANNEL PL 1/2 x 5 x AS REQ'D $W_t = 8.52$ PLF	
(2) SIDE FRAMING PLs 1/8 x 6 1/4 x AS REQ'D $W_t = 5.33$ PLF	
ASSUMING 12 FT LONG x 5.5 FT WIDE x 17" DEEP CONCRETE BEAM, w/ $\gamma_{concrete} = 150$ PCF	
$DL_{concrete} = \left[(5.5')\left(\frac{17''}{12''/ft}\right) - (5 \text{ pockets})\left(\frac{5''}{12''/ft} \times \frac{9''}{12''/ft}\right)\right]\gamma_{concrete}$	
$= (7.792 \text{ ft}^2 - 1.56 \text{ ft}^2)(150 \text{ PCF}) = 935$ PLF	
BRAKING FORCE (BR)	AASHTO LRFD 3.6.4
TAKEN AS 25% OF AXLES WEIGHTS OF DESIGN TRUCK (32^k AXLE LL)	
25% OF AXLE WEIGHTS OF DESIGN TANDEM (25^k AXLES LL)	
- OR -	
5% OF OF EITHER + LANE LOAD (640 PLF) +	
$BR = 0.25(32^k) = 8^k$/AXLE $= 4^k$/TIRE AREA (25% OF TRUCK)	
$= 0.25(25^k) = 6.25^k$/AXLE $= 3.13$/TIRE AREA (25% OF TANDEM)	
$= 0.05(32^k) = 1.6^k$/AXLE (5% OF EITHER + LANE)	
$= 0.8^k$/TIRE AREA $+ 0.64^k = 0.72^k$/TIRE AREA	
THE 25% OF DESIGN TRUCK GOVERNS 4K/TIRE AREA, PRESENTED AS A PSF HORIZONTAL FORCE, $BR = \dfrac{4K(1000\#/k)}{(20'')(10'')/144} = 2880$ PSF	

5 squares per inch

MWH now part of Stantec

By ______ Date 1/30/18 Client UNLV Sheet 3 of ____

Chkd. By ______ Description ______ Job No. 181307096

Design Task ______

THE LOAD COMBINATIONS PER AASHTO:

STRENGTH I: $1.25(DC) + 1.75(LL + IM + BR)$ ← GOVERNS FACTORED DESIGN

SERVICE II: $1.00(DC + 1.30(LL + IM + BR)$

LOAD COMBINATIONS PER ACI 318-11:

$1.4D$ ← GOVERNS FOR DL ONLY

$1.2D + 1.6L$

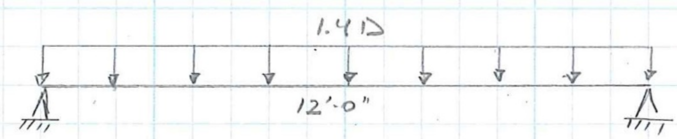
REFERENCES

AASHTO LRFD TABLE 3.4.1-1

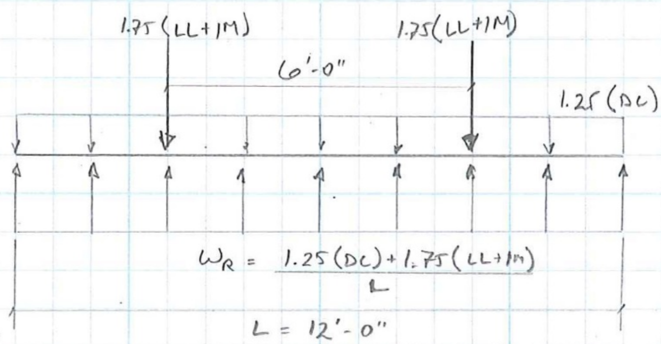
ACI 318 9.2.1

LOAD SCENARIOS

CASE 1: RUMBLE STRIP IS CRANED, LOADING IS DUE TO DEAD LOAD ONLY, REACTIONS AT ANCHOR POINTS (PINNED CONNECTION), 1.4DL GOVERNS



CASE 2: RUMBLE STRIP IS INSTALLED AND SUBJECTED TO TRAFFIC LOADING. SOIL PUSHES UP AGAINST BOX DUE TO TIRE BEARING PRESSURE AND BOX SELF WT. GOVERNING FACTORED LOAD IS: $1.25(DC) + 1.75(LL + IM + BR)$



5 squares per inch

β' = slope of ground surface in front of wall {+ for slope up from wall; – for slope down from wall} (degrees) (3.11.5.6)
γ = load factors; unit weight of materials (kcf); unit weight of water (kcf); unit weight of soil (kcf) (C3.4.1) (3.5.1) (C3.9.5) (3.11.5.1)
γ_s = unit weight of soil (kcf) (3.11.5.1)
γ'_s = effective soil unit weight (kcf) (3.11.5.6)
γ_{EQ} = load factor for live load applied simultaneously with seismic loads (3.4.1)
γ_{eq} = equivalent-fluid unit weight of soil (kcf) (3.11.5.5)
γ_i = load factor (3.4.1)
γ_p = load factor for permanent loading (3.4.1)
γ_{SE} = load factor for settlement (3.4.1)
γ_{TG} = load factor for temperature gradient (3.4.1)
Δ = movement of top of wall required to reach minimum active or maximum passive pressure by tilting or lateral translation (ft) (C3.11.1) (3.11.5.5)
Δ_p = constant horizontal earth pressure due to uniform surcharge (ksf) (3.11.6.1)
Δ_{ph} = constant horizontal pressure distribution on wall resulting from various types of surcharge loading (ksf) (3.11.6.2)
Δ_T = design thermal movement range (in.) (3.12.2.3)
$\Delta\sigma_H$ = horizontal stress due to surcharge load (ksf) (3.11.6.3)
$\Delta\sigma_v$ = vertical stress due to surcharge load (ksf) (3.11.6.3)
δ = angle of truncated ice wedge (degrees); friction angle between fill and wall (degrees); angle between foundation wall and a line connecting the point on the wall under consideration and a point on the bottom corner of the footing furthest from the wall (rad) (C3.9.5) (3.11.5.3) (3.11.6.2)
η_i = load modifier specified in Article 1.3.2; wall face batter (3.4.1) (3.11.5.9)
θ = angle of back of wall to the horizontal (degrees); angle of channel turn or bend (degrees); angle between direction of stream flow and the longitudinal axis of pier (degrees) (3.11.5.3) (3.14.5.2.3) (3.7.3.2)
θ_f = friction angle between ice floe and pier (degrees) (3.9.2.4.1)
σ = standard deviation of normal distribution (3.14.5.3)
σ_T = tensile strength of ice (ksf) (C3.9.5)
ν = Poisson's Ratio (dim.) (3.11.6.2)
ϕ = resistance factors (C3.4.1)
ϕ_f = angle of internal friction (degrees) (3.11.5.4)
ϕ'_f = effective angle of internal friction (degrees) (3.11.5.2)
ϕ_r = internal friction angle of reinforced fill (degrees) (3.11.6.3)
ϕ'_s = angle of internal friction of retained soil (degrees) (3.11.5.6)

3.3.2—Load and Load Designation

The following permanent and transient loads and forces shall be considered:

- Permanent Loads

CR = force effects due to creep
DD = downdrag force
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EH = horizontal earth pressure load
EL = miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill

PS = secondary forces from post-tensioning
SH = force effects due to shrinkage

- Transient Loads

BL = blast loading
BR = vehicular braking force
CE = vehicular centrifugal force
CT = vehicular collision force
CV = vessel collision force
EQ = earthquake load
FR = friction load
IC = ice load
IM = vehicular dynamic load allowance
LL = vehicular live load
LS = live load surcharge
PL = pedestrian live load
SE = force effect due to settlement
TG = force effect due to temperature gradient
TU = force effect due to uniform temperature
WA = water load and stream pressure
WL = wind on live load
WS = wind load on structure

3.4—LOAD FACTORS AND COMBINATIONS

3.4.1—Load Factors and Load Combinations

The total factored force effect shall be taken as:

$$Q = \Sigma \eta_i \gamma_i Q_i \quad (3.4.1\text{-}1)$$

where:

η_i = load modifier specified in Article 1.3.2
Q_i = force effects from loads specified herein
γ_i = load factors specified in Tables 3.4.1-1 and 3.4.1-2

Components and connections of a bridge shall satisfy Eq. 1.3.2.1-1 for the applicable combinations of factored extreme force effects as specified at each of the following limit states:

- Strength I—Basic load combination relating to the normal vehicular use of the bridge without wind.
- Strength II—Load combination relating to the use of the bridge by Owner-specified special design vehicles, evaluation permit vehicles, or both without wind.
- Strength III—Load combination relating to the bridge exposed to wind velocity exceeding 55 mph.

C3.4.1

The background for the load factors specified herein, and the resistance factors specified in other Sections of these Specifications is developed in Nowak (1992).

The permit vehicle should not be assumed to be the only vehicle on the bridge unless so assured by traffic control. See Article 4.6.2.2.5 regarding other traffic on the bridge simultaneously.

Vehicles become unstable at higher wind velocities. Therefore, high winds prevent the presence of significant live load on the bridge.

The load factor for settlement, γ_{SE}, should be considered on a project-specific basis. In lieu of project-specific information to the contrary, γ_{SE}, may be taken as 1.0. Load combinations which include settlement shall also be applied without settlement.

For segmentally constructed bridges, the following combination shall be investigated at the service limit state:

$$DC + DW + EH + EV + ES + WA + CR + SH + TG + EL + PS \quad (3.4.1\text{-}2)$$

Table 3.4.1-1—Load Combinations and Load Factors

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time *EQ*	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—	—
Fatigue I—*LL*, *IM* & *CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II—*LL*, *IM* & *CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—	—

Table 3.4.1-2—Load Factors for Permanent Loads, γ_p

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor Maximum	Load Factor Minimum
DC: Component and Attachments		1.25	0.90
DC: Strength IV only		1.50	0.90
DD: Downdrag	Piles, α Tomlinson Method	1.4	0.25
	Piles, λ Method	1.05	0.30
	Drilled shafts, O'Neill and Reese (1999) Method	1.25	0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure			
• Active		1.50	0.90
• At-Rest		1.35	0.90
• *AEP* for anchored walls		1.35	N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure			
• Overall Stability		1.00	N/A
• Retaining Walls and Abutments		1.35	1.00
• Rigid Buried Structure		1.30	0.90
• Rigid Frames		1.35	0.90
• Flexible Buried Structures			
o Metal Box Culverts and Structural Plate Culverts with Deep Corrugations		1.5	0.9
o Thermoplastic culverts		1.3	0.9
o All others		1.95	0.9
ES: Earth Surcharge		1.50	0.75

Table 3.4.1-3—Load Factors for Permanent Loads Due to Superimposed Deformations, γ_p

Bridge Component	*PS*	*CR, SH*
Superstructures—Segmental Concrete Substructures supporting Segmental Superstructures (see 3.12.4, 3.12.5)	1.0	See γ_P for *DC*, Table 3.4.1-2
Concrete Superstructures—non-segmental	1.0	1.0
Substructures supporting non-segmental Superstructures		
• using I_g	0.5	0.5
• using $I_{effective}$	1.0	1.0
Steel Substructures	1.0	1.0

The multiple presence factors in Table 3.6.1.1.2-1 were developed on the basis of an ADTT of 5,000 trucks in one direction. The force effect resulting from the appropriate number of lanes may be reduced for sites with lower ADTT as follows:

- If $100 \leq ADTT \leq 1{,}000$, 95 percent of the specified force effect may be used; and
- If $ADTT < 100$, 90 percent of the specified force effect may be used.

This adjustment is based on the reduced probability of attaining the design event during a 75-year design life with reduced truck volume.

3.6.1.2—Design Vehicular Live Load

3.6.1.2.1—General

Vehicular live loading on the roadways of bridges or incidental structures, designated HL-93, shall consist of a combination of the:

- Design truck or design tandem, and
- Design lane load.

Except as modified in Article 3.6.1.3.1, each design lane under consideration shall be occupied by either the design truck or tandem, coincident with the lane load, where applicable. The loads shall be assumed to occupy 10.0 ft transversely within a design lane.

C3.6.1.2.1

Consideration should be given to site-specific modifications to the design truck, design tandem, and/or the design lane load under the following conditions:

- The legal load of a given jurisdiction is significantly greater than typical;
- The roadway is expected to carry unusually high percentages of truck traffic;
- Flow control, such as a stop sign, traffic signal, or toll booth, causes trucks to collect on certain areas of a bridge or to not be interrupted by light traffic; or
- Special industrial loads are common due to the location of the bridge.

See also discussion in Article C3.6.1.3.1.

The live load model, consisting of either a truck or tandem coincident with a uniformly distributed load, was developed as a notional representation of shear and moment produced by a group of vehicles routinely permitted on highways of various states under "grandfather" exclusions to weight laws. The vehicles considered to be representative of these exclusions were based on a study conducted by the Transportation Research Board (Cohen, 1990). The load model is called "notional" because it is not intended to represent any particular truck.

In the initial development of the notional live load model, no attempt was made to relate to escorted permit loads, illegal overloads, or short duration special permits. The moment and shear effects were subsequently compared to the results of truck weight studies (Csagoly and Knobel, 1981; Nowak, 1992), selected WIM data, and the 1991 OHBDC live load model. These subsequent comparisons showed that the notional load could be scaled by appropriate load factors to be representative of these other load spectra.

The following nomenclature applies to Figures C3.6.1.2.1-1 through C3.6.1.2.1-6, which show results of live load studies involving two equal continuous spans or simple spans:

M POS 0.4L	=	positive moment at 4/10 point in either span
M NEG 0.4L	=	negative moment at 4/10 point in either span
M SUPPORT	=	moment at interior support
Vab	=	shear adjacent to either exterior support
Vba	=	shear adjacent to interior support
Mss	=	midspan moment in a simply supported span

The "span" is the length of the simple-span or of one of each of the two continuous spans. The comparison is in the form of ratios of the load effects produced in either simple-span or two-span continuous girders. A ratio greater than 1.0 indicates that one or more of the exclusion vehicles produces a larger load effect than the HS20 loading. The figures indicate the degree by which the exclusion loads deviate from the HS loading of designation, e.g., HS25.

Figures C3.6.1.2.1-1 and C3.6.1.2.1-2 show moment and shear comparisons between the envelope of effects caused by 22 truck configurations chosen to be representative of the exclusion vehicles and the HS20 loading, either the HS20 truck or the lane load, or the interstate load consisting of two 24.0-kip axles 4.0 ft apart, as used in previous editions of the AASHTO Standard Specifications. The largest and smallest of the 22 configurations can be found in Kulicki and Mertz (1991). In the case of negative moment at an interior support, the results presented are based on two identical exclusion vehicles in tandem and separated by at least 50.0 ft.

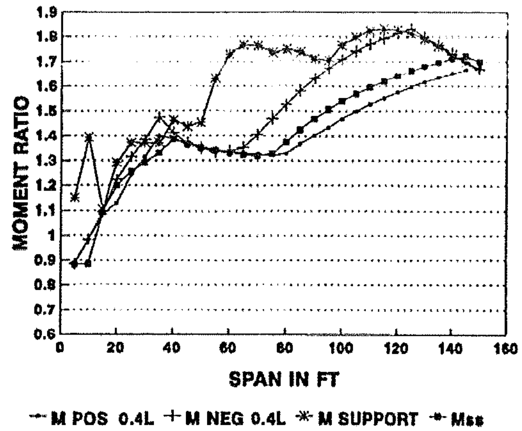


Figure C3.6.1.2.1-1—Moment Ratios: Exclusion Vehicles to HS20 (truck or lane) or Two 24.0-kip Axles at 4.0 ft

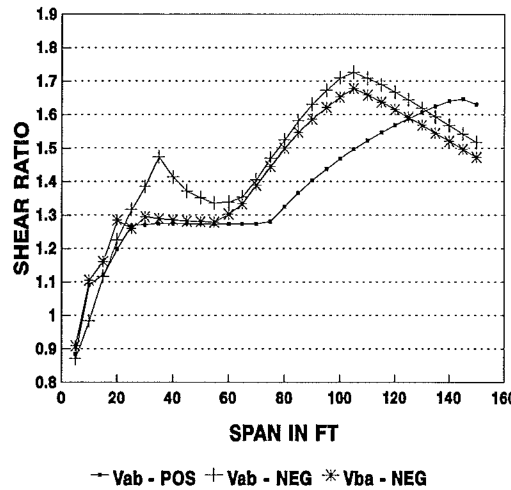


Figure C3.6.1.2.1-2—Shear Ratios: Exclusion Vehicles to HS20 (truck or lane) or Two 24.0-kip Axles at 4.0 ft

Figures C3.6.1.2.1-3 and C3.6.1.2.1-4 show comparisons between the force effects produced by a single exclusion truck per lane and the notional load model, except for negative moment, where the tandem exclusion vehicles were used. In the case of negative moment at a support, the provisions of Article 3.6.1.3.1 requiring investigation of 90 percent of the effect of two design trucks, plus 90 percent of the design lane load, has been included in Figures C3.6.1.2.1-3 and C3.6.1.2.1-5. Compared with Figures C3.6.1.2.1-1 and C3.6.1.2.1-2, the range of ratios can be seen as more closely grouped:

- Over the span range,
- Both for shear and moment, and
- Both for simple-span and continuous spans.

The implication of close grouping is that the notional load model with a single-load factor has general applicability.

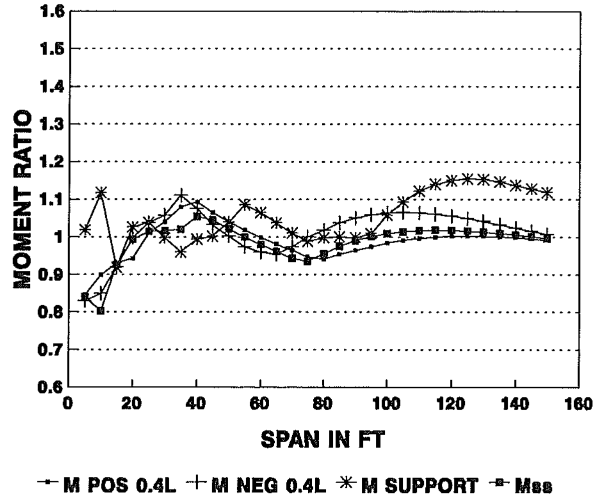


Figure C3.6.1.2.1-3—Moment Ratios: Exclusion Vehicles to Notional Model

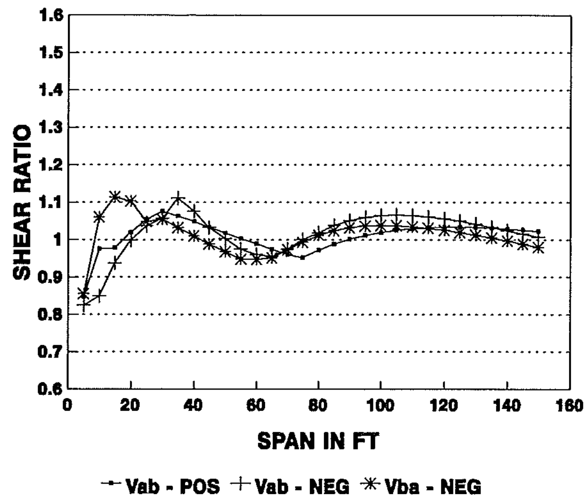


Figure C3.6.1.2.1-4—Shear Ratios: Exclusion Vehicles to Notional Model

Figures C3.6.1.2.1-5 and C3.6.1.2.1-6 show the ratios of force effects produced by the notional load model and the greatest of the HS20 truck or lane loading, or Alternate Military Loading.

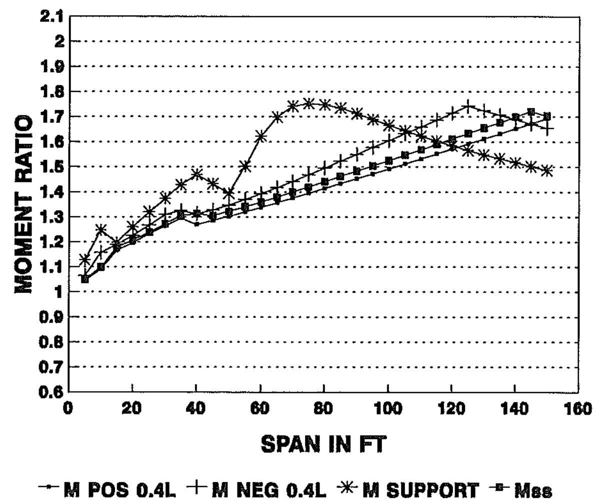


Figure C3.6.1.2.1-5—Moment Ratios: Notional Model to HS20 (truck or lane) or Two 24.0-kip Axles at 4.0 ft

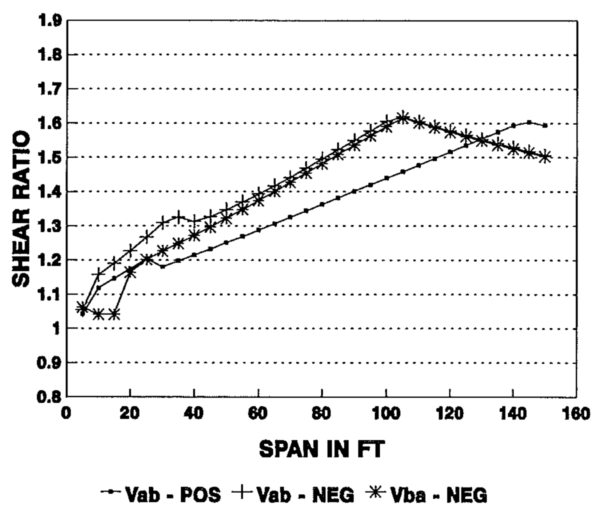


Figure C3.6.1.2.1-6—Shear Ratios: Notional Model to HS20 (truck and lane) or Two 24.0-kip Axles at 4.0 ft

In reviewing Figures C3.6.1.2.1-5 and C3.6.1.2.1-6, it should be noted that the total design force effect is also a function of load factor, load modifier, load distribution, and dynamic load allowance.

3.6.1.2.2—Design Truck

The weights and spacings of axles and wheels for the design truck shall be as specified in Figure 3.6.1.2.2-1. A dynamic load allowance shall be considered as specified in Article 3.6.2.

Except as specified in Articles 3.6.1.3.1 and 3.6.1.4.1, the spacing between the two 32.0-kip axles shall be varied between 14.0 ft and 30.0 ft to produce extreme force effects.

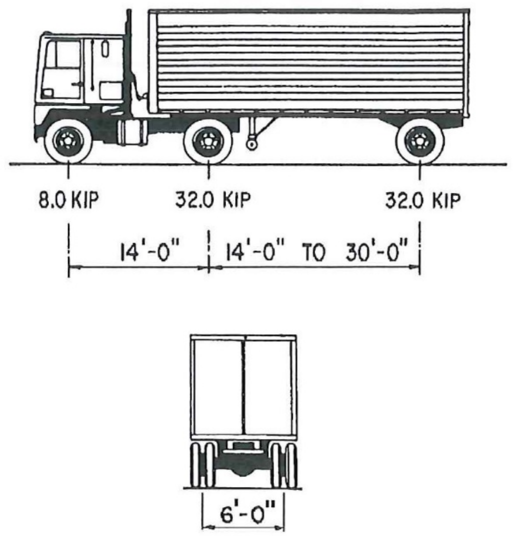


Figure 3.6.1.2.2-1—Characteristics of the Design Truck

3.6.1.2.3—Design Tandem

The design tandem shall consist of a pair of 25.0-kip axles spaced 4.0 ft apart. The transverse spacing of wheels shall be taken as 6.0 ft. A dynamic load allowance shall be considered as specified in Article 3.6.2.

3.6.1.2.4—Design Lane Load

The design lane load shall consist of a load of 0.64 klf uniformly distributed in the longitudinal direction. Transversely, the design lane load shall be assumed to be uniformly distributed over a 10.0-ft width. The force effects from the design lane load shall not be subject to a dynamic load allowance.

3.6.1.2.5—Tire Contact Area

The tire contact area of a wheel consisting of one or two tires shall be assumed to be a single rectangle, whose width is 20.0 in. and whose length is 10.0 in.

The tire pressure shall be assumed to be uniformly distributed over the contact area. The tire pressure shall be assumed to be distributed as follows:

- On continuous surfaces, uniformly over the specified contact area, and
- On interrupted surfaces, uniformly over the actual contact area within the footprint with the pressure increased in the ratio of the specified to actual contact areas.

For the design of orthotropic decks and wearing surfaces on orthotropic decks, the front wheels shall be assumed to be a single rectangle whose width and length are both 10.0 in. as specified in Article 3.6.1.4.1.

C3.6.1.2.5

The area load applies only to the design truck and tandem. For other design vehicles, the tire contact area should be determined by the engineer.

As a guideline for other truck loads, the tire area in in.^{2} may be calculated from the following dimensions:

Tire width = $P/0.8$

Tire length = $6.4\gamma(1 + IM/100)$

where:

γ = load factor
IM = dynamic load allowance percent
P = design wheel load (kip)

3.6.1.2.6—Distribution of Wheel Loads through Earth Fills

Where the depth of fill is less than 2.0 ft, live loads shall be distributed to the top slabs of culverts as specified in Article 4.6.2.10.

In lieu of a more precise analysis, or the use of other acceptable approximate methods of load distribution permitted in Section 12, where the depth of fill is 2.0 ft or greater, wheel loads may be considered to be uniformly distributed over a rectangular area with sides equal to the dimension of the tire contact area, as specified in Article 3.6.1.2.5, and increased by either 1.15 times the depth of the fill in select granular backfill, or the depth of the fill in all other cases. The provisions of Articles 3.6.1.1.2 and 3.6.1.3 shall apply.

Where such areas from several wheels overlap, the total load shall be uniformly distributed over the area.

For single-span culverts, the effects of live load may be neglected where the depth of fill is more than 8.0 ft and exceeds the span length; for multiple span culverts, the effects may be neglected where the depth of fill exceeds the distance between faces of end walls.

Where the live load and impact moment in concrete slabs, based on the distribution of the wheel load through earth fills, exceeds the live load and impact moment calculated according to Article 4.6.2.10, the latter moment shall be used.

C3.6.1.2.6

Elastic solutions for pressures produced within an infinite half-space by loads on the ground surface can be found in Poulos and Davis (1974), NAVFAC DM-7.1 (1982), and soil mechanics textbooks.

This approximation is similar to the 60-degree rule found in many texts on soil mechanics. The dimensions of the tire contact area are determined at the surface based on the dynamic load allowance of 33 percent at depth = 0. They are projected through the soil as specified. The pressure intensity on the surface is based on the wheel load without dynamic load allowance. A dynamic load allowance is added to the pressure on the projected area. The dynamic load allowance also varies with depth as specified in Article 3.6.2.2. The design lane load is applied where appropriate and multiple presence factors apply.

This provision applies to relieving slabs below grade and to top slabs of box culverts.

Traditionally, the effect of fills less than 2.0 ft deep on live load has been ignored. Research (McGrath, et al. 2004) has shown that in design of box sections allowing distribution of live load through fill in the direction parallel to the span provides a more accurate design model to predict moment, thrust, and shear forces. Provisions in Article 4.6.2.10 provide a means to address the effect of shallow fills.

3.6.1.3—Application of Design Vehicular Live Loads

3.6.1.3.1—General

Unless otherwise specified, the extreme force effect shall be taken as the larger of the following:

- The effect of the design tandem combined with the effect of the design lane load, or
- The effect of one design truck with the variable axle spacing specified in Article 3.6.1.2.2, combined with the effect of the design lane load, and

C3.6.1.3.1

The effects of an axle sequence and the lane load are superposed in order to obtain extreme values. This is a deviation from the traditional AASHTO approach, in which either the truck or the lane load, with an additional concentrated load, provided for extreme effects.

The lane load is not interrupted to provide space for the axle sequences of the design tandem or the design truck; interruption is needed only for patch loading patterns to produce extreme force effects.

Table 3.6.2.1-1—Dynamic Load Allowance, *IM*

Component	IM
Deck Joints—All Limit States	75%
All Other Components:	
• Fatigue and Fracture Limit State	15%
• All Other Limit States	33%

The application of dynamic load allowance for buried components, covered in Section 12, shall be as specified in Article 3.6.2.2.

Dynamic load allowance need not be applied to:

- Retaining walls not subject to vertical reactions from the superstructure, and
- Foundation components that are entirely below ground level.

The dynamic load allowance may be reduced for components, other than joints, if justified by sufficient evidence, in accordance with the provisions of Article 4.7.2.1.

- Dynamic response of the bridge as a whole to passing vehicles, which may be due to long undulations in the roadway pavement, such as those caused by settlement of fill, or to resonant excitation as a result of similar frequencies of vibration between bridge and vehicle.

Field tests indicate that in the majority of highway bridges, the dynamic component of the response does not exceed 25 percent of the static response to vehicles. This is the basis for dynamic load allowance with the exception of deck joints. However, the specified live load combination of the design truck and lane load, represents a group of exclusion vehicles that are at least 4/3 of those caused by the design truck alone on short- and medium-span bridges. The specified value of 33 percent in Table 3.6.2.1-1 is the product of 4/3 and the basic 25 percent.

Generally speaking, the dynamic amplification of trucks follows the following general trends:

- As the weight of the vehicle goes up, the apparent amplification goes down.
- Multiple vehicles produce a lower dynamic amplification than a single vehicle.
- More axles result in a lower dynamic amplification.

For heavy permit vehicles which have many axles compared to the design truck, a reduction in the dynamic load allowance may be warranted. A study of dynamic effects presented in a report by the Calibration Task Group (*Nowak 1992*) contains details regarding the relationship between dynamic load allowance and vehicle configuration.

This Article recognizes the damping effect of soil when in contact with some buried structural components, such as footings. To qualify for relief from impact, the entire component must be buried. For the purpose of this Article, a retaining type component is considered to be buried to the top of the fill.

3.6.2.2—Buried Components

The dynamic load allowance for culverts and other buried structures covered by Section 12, in percent, shall be taken as:

$$IM = 33(1.0 - 0.125D_E) \geq 0\% \qquad (3.6.2.2\text{-}1)$$

where:

D_E = the minimum depth of earth cover above the structure (ft)

3.6.2.3—Wood Components

Dynamic load allowance need not be applied to wood components.

C3.6.2.3

Wood structures are known to experience reduced dynamic wheel load effects due to internal friction between the components and the damping characteristics of wood. Additionally, wood is stronger for short duration loads, as compared to longer duration loads. This increase in strength is greater than the increase in force effects resulting from the dynamic load allowance.

3.6.3—Centrifugal Forces: *CE*

For the purpose of computing the radial force or the overturning effect on wheel loads, the centrifugal effect on live load shall be taken as the product of the axle weights of the design truck or tandem and the factor *C*, taken as:

$$C = f\frac{v^2}{gR} \tag{3.6.3-1}$$

where:

v = highway design speed (ft/s)
f = 4/3 for load combinations other than fatigue and 1.0 for fatigue
g = gravitational acceleration: 32.2 (ft/s^2)
R = radius of curvature of traffic lane (ft)

Highway design speed shall not be taken to be less than the value specified in the current edition of the AASHTO publication, *A Policy of Geometric Design of Highways and Streets*.

The multiple presence factors specified in Article 3.6.1.1.2 shall apply.

Centrifugal forces shall be applied horizontally at a distance 6.0 ft above the roadway surface. A load path to carry the radial force to the substructure shall be provided.

The effect of superelevation in reducing the overturning effect of centrifugal force on vertical wheel loads may be considered.

C3.6.3

Centrifugal force is not required to be applied to the design lane load, as the spacing of vehicles at high speed is assumed to be large, resulting in a low density of vehicles following and/or preceding the design truck. For all other consideration of live load other than for fatigue, the design lane load is still considered even though the centrifugal effect is not applied to it.

The specified live load combination of the design truck and lane load, however, represents a group of exclusion vehicles that produce force effects of at least 4/3 of those caused by the design truck alone on short- and medium-span bridges. This ratio is indicated in Eq. 3.6.3-1 for the service and strength limit states. For the fatigue and fracture limit state, the factor 1.0 is consistent with cumulative damage analysis. The provision is not technically perfect, yet it reasonably models the representative exclusion vehicle traveling at design speed with large headways to other vehicles. The approximation attributed to this convenient representation is acceptable in the framework of the uncertainty of centrifugal force from random traffic patterns.

1.0 ft/s = 0.682 mph

Centrifugal force also causes an overturning effect on the wheel loads because the radial force is applied 6.0 ft above the top of the deck. Thus, centrifugal force tends to cause an increase in the vertical wheel loads toward the outside of the bridge and an unloading of the wheel loads toward the inside of the bridge. Superelevation helps to balance the overturning effect due to the centrifugal force and this beneficial effect may be considered. The effects due to vehicle cases with centrifugal force effects included should be compared to the effects due to vehicle cases with no centrifugal force, and the worst case selected.

3.6.4—Braking Force: *BR*

The braking force shall be taken as the greater of:

- 25 percent of the axle weights of the design truck or design tandem or,
- Five percent of the design truck plus lane load or five percent of the design tandem plus lane load

C3.6.4

Based on energy principles, and assuming uniform deceleration, the braking force determined as a fraction of vehicle weight is:

$$b = \frac{v^2}{2ga} \tag{C3.6.4-1}$$

This braking force shall be placed in all design lanes which are considered to be loaded in accordance with Article 3.6.1.1.1 and which are carrying traffic headed in the same direction. These forces shall be assumed to act horizontally at a distance of 6.0 ft above the roadway surface in either longitudinal direction to cause extreme force effects. All design lanes shall be simultaneously loaded for bridges likely to become one-directional in the future.

The multiple presence factors specified in Article 3.6.1.1.2 shall apply.

where a is the length of uniform deceleration and b is the fraction. Calculations using a braking length of 400 ft and a speed of 55 mph yield $b = 0.25$ for a horizontal force that will act for a period of about 10 s. The factor b applies to all lanes in one direction because all vehicles may have reacted within this time frame.

For short- and medium-span bridges, the specified braking force can be significantly larger than was required in the Standard Specifications. The braking force specified in the Standard Specifications dates back to at least the early 1940's without any significant changes to address the improved braking capacity of modern trucks. A review of other bridge design codes in Canada and Europe showed that the braking force required by the Standard Specification is much lower than that specified in other design codes for most typical bridges. One such comparison is shown in Figure C3.6.4-1.

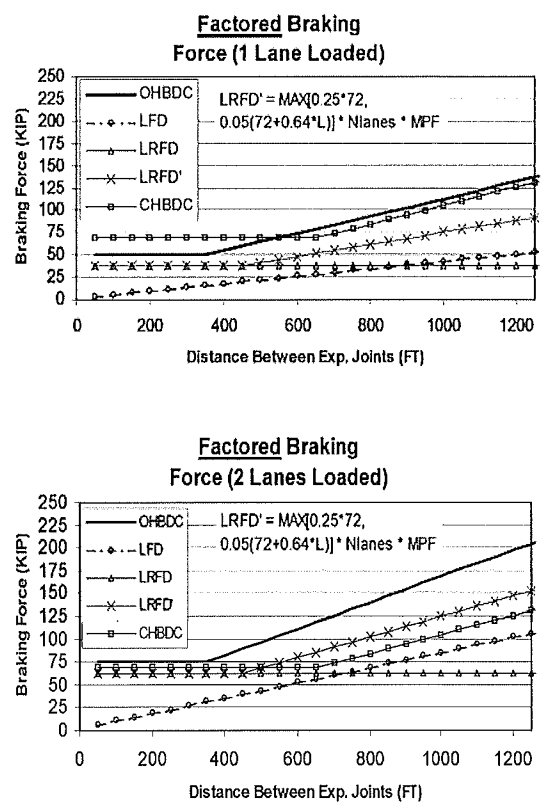


Figure C3.6.4-1—Comparison of Braking Force Models

SECTION 200
FOUNDATION
BEAM AND SLAB

By J. BATILOV Date 1/31/18 Client UNLV Sheet 9 of

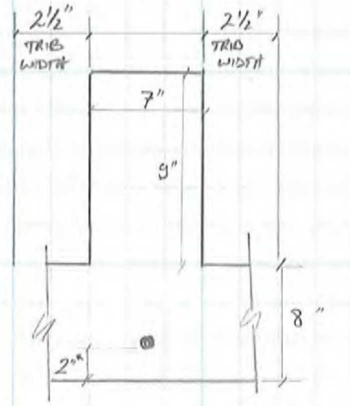
Chkd. By D. ROUNDS Description DRT RUMBLE STRIP Job No. 181307096

Design Task

REFERENCES

LOADING ON 7" WIDE CONCRETE STRIP BETWEEN MECHANISM POCKETS

THE CONCRETE STRIP BEAM BETWEEN EACH MECHANISM WILL BE DESIGNED AS A STAND ALONE BEAM TO SIZE LONGITUDINAL REINFORCEMENT



* 2" COVER PER ACI 318-11 7.7.1 SINCE THIS SIDE WILL BE IN CONTACT W/ EARTH BUT NOT CAST AGAINST IT

LOADING

DEAD LOAD (DC)

ASSUMING THE BEAM ON THE RIGHT IS SOLID CONCRETE INCLUDING THE 5" TRIB WIDTH

$$DL_{CONC} = \left(\frac{7"+5"}{12"/_{FT}}\right)\left(\frac{9"+8"}{12"/_{FT}}\right)(150\ PCF)$$

$= 230\ PLF$

$1.25\,DL = 1.25\,(230\ PLF) = 288\ PLF$

TRAFFIC LOAD + IM (LL+IM)

HL-93 DESIGN TRUCK (32^K AXLE) GOVERNED, EQUIVALENT TO 16^K WHEEL LOAD

$LL+IM = 1.33\,(16^K) = 21.28\,K\,/\,TIRE.$

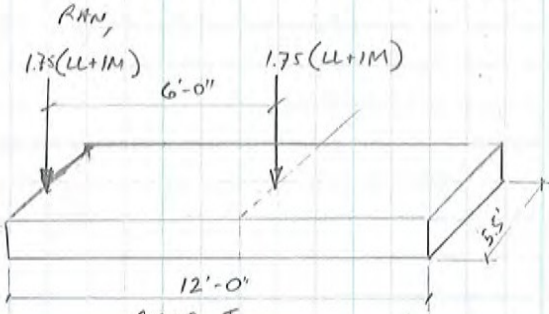
$1.75(LL+IM) = 1.75\,(21.28^K) = 37.24$

THE ANALYSIS OF THE RUMBLE STRIP BOX HAS BEEN ANALYZED IN SAFE 2016 FOUNDATION ANALYSIS PROGRAM.

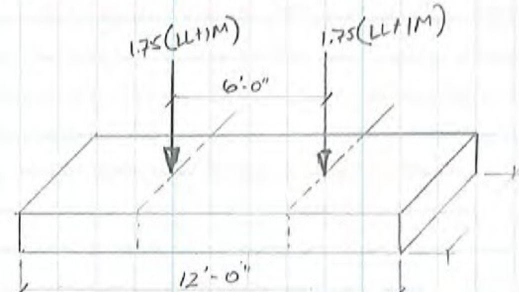
BOX MODELED AS 8" MATT 12'-0" x 5'-6" w/ 9" HIGH STEM WALLS

THE SOIL SUBGRADE MODULUS SELECTED IS AT THE LOW END OF AASHTO FAIR ROADBED SOIL QUALITY.

TWO CASES OF THE LOAD COMBINATION $1.25D + 1.75(LL+IM)$ WERE RAN,



CASE I



CASE II

5 squares per inch

MWH now part of Stantec

By I. BATTLOU Date 2/23/18 Client UNLV Sheet 10 of
Chkd. By D. ROUNDS Description DRT RUMBLE STRIP Job No. 181307096
Design Task

REFERENCES

BASED ON THE SAFE ANALYSIS THE FOLLOWING ARE THE FLEXURAL & SHEAR DEMANDS, NOTE DESIGN STRIPS ARE 6" ALONG X-AXIS / 12" ALONG Y-AXIS

SAFE ANALYSIS FIGURES

ALONG THE 12'-0" LENGTH OF THE BOX (X·AXIS) - 6 INCH DESIGN STRIPS

M_u = 0.9847 K·FT (STRENGTH II LC - SEE FIG. 17) FOR 6" DESIGN STRIP
" = 1.9694 K-FT FOR A 12" DESIGN STRIP
V_u = 0.771 K (STRENGTH I LC - SEE FIG. 14) FOR 6" DESIGN STRIP
= 1.542 K FOR A 12" DESIGN STRIP

ALONG THE 5'-7" WIDTH OF THE BOX (Y·AXIS), - 12" DESIGN STRIPS
M_u = 8.001 K·FT (STRENGTH I LC - FIGURE 15)
V_u = 6.249 K (STRENGTH I LC - FIGURE 16)

5 squares per inch

UNLV DRT Rumble Strip - SAFE 2016 Analysis Output

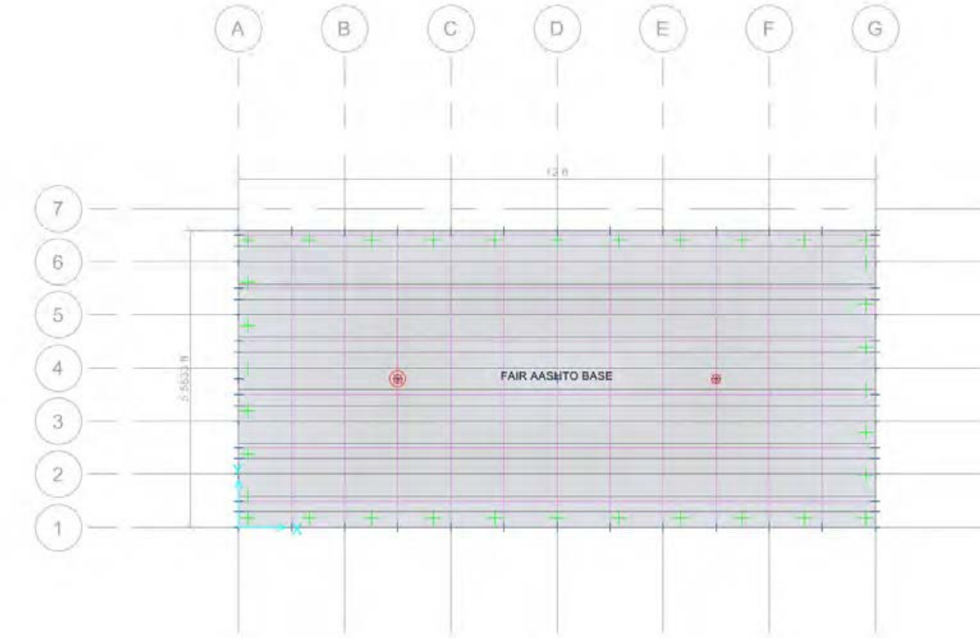
Box Dimensions

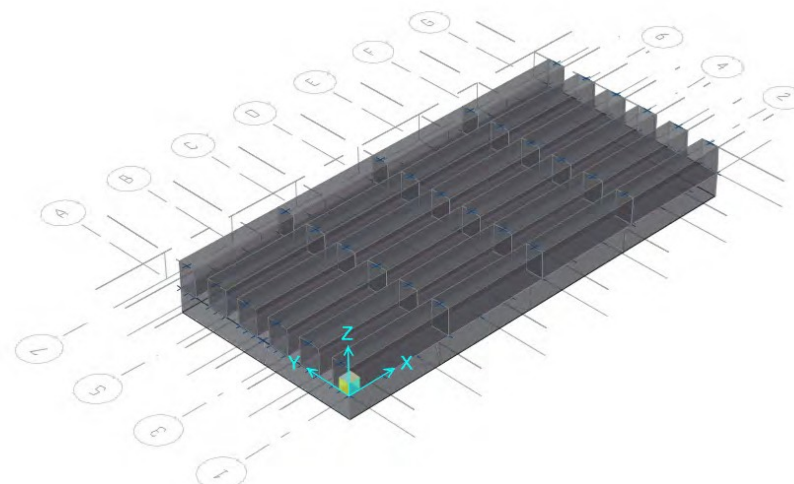
8" Base Slab 12'-0" x 5'-6"

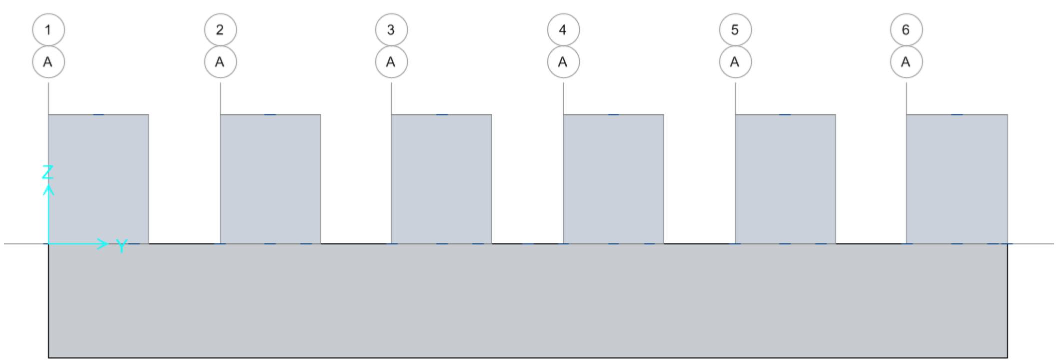
7"W x 9"H Stem Walls

Materials: 4500 psi Concrete

A615 Gr.60 Reinforcement







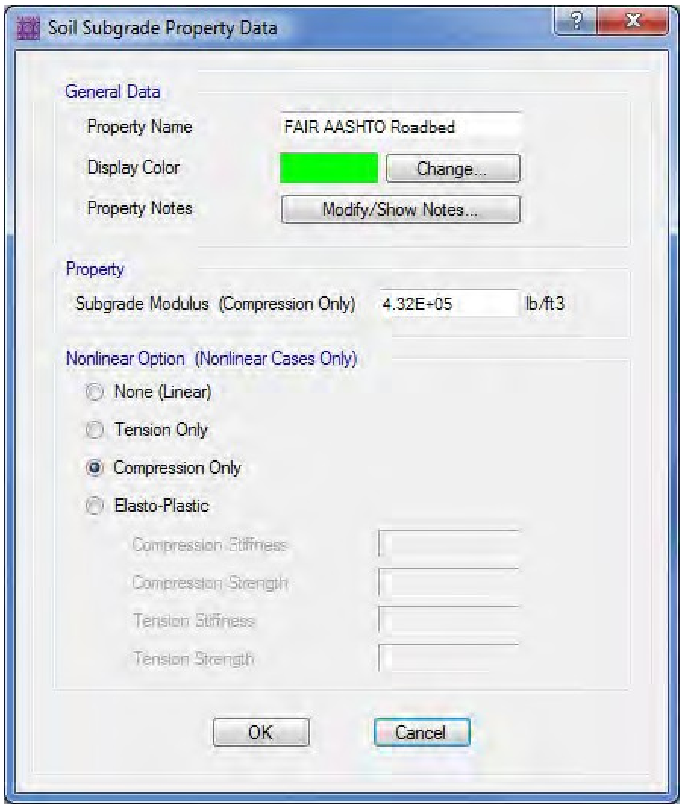
1 Soil Subgrade Properties

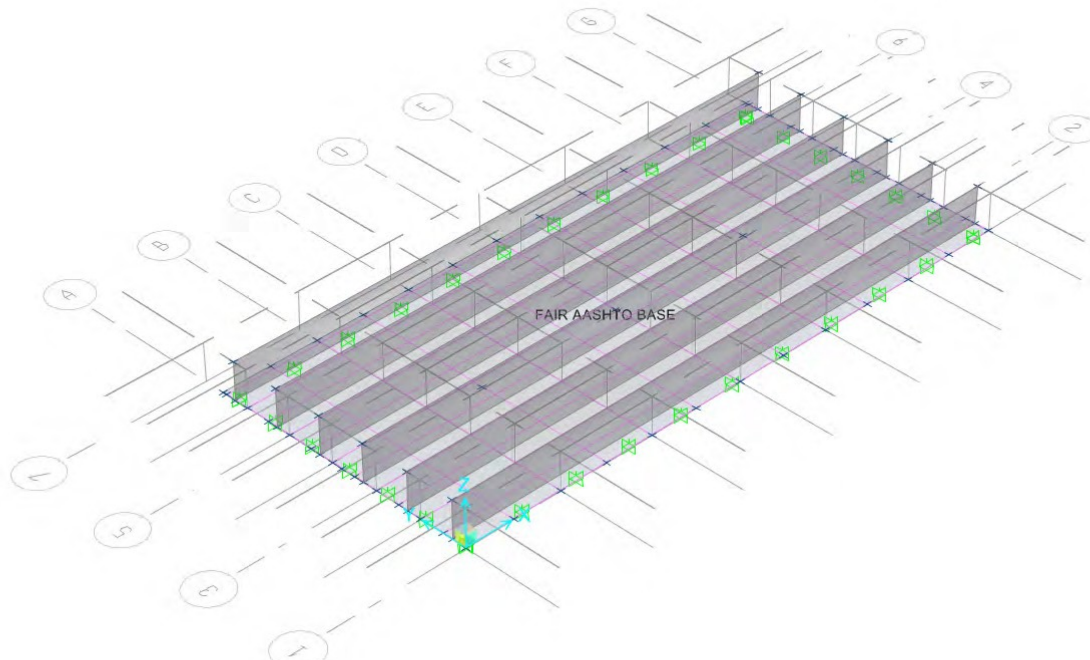
Chosen from ***NHI-05-037 - Geotech - Bridges & Structures - Federal Highway Administration***

Conservatively the lower bound of **FAIR** roadbed soil quality selected: 250 lbf/in^2/in = 432000 lbf/ft^2/ft

Table 5-46. Suggested ranges for modulus of subgrade reaction for design (AASHTO, 1993).

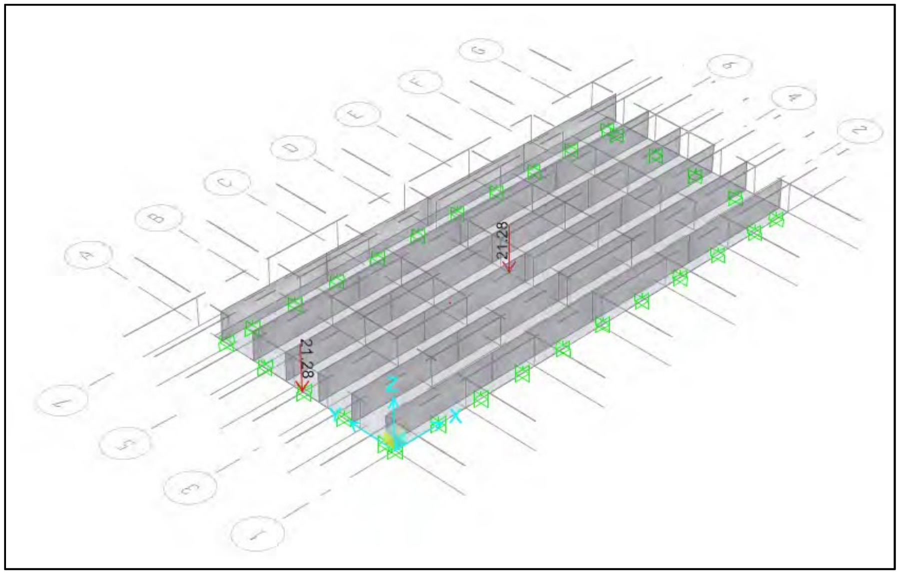
Roadbed Soil Quality	Range for k_{eff} (pci)
Very Good	> 550
Good	400 - 500
Fair	250 - 350
Poor	150 - 250
Very Poor	< 150



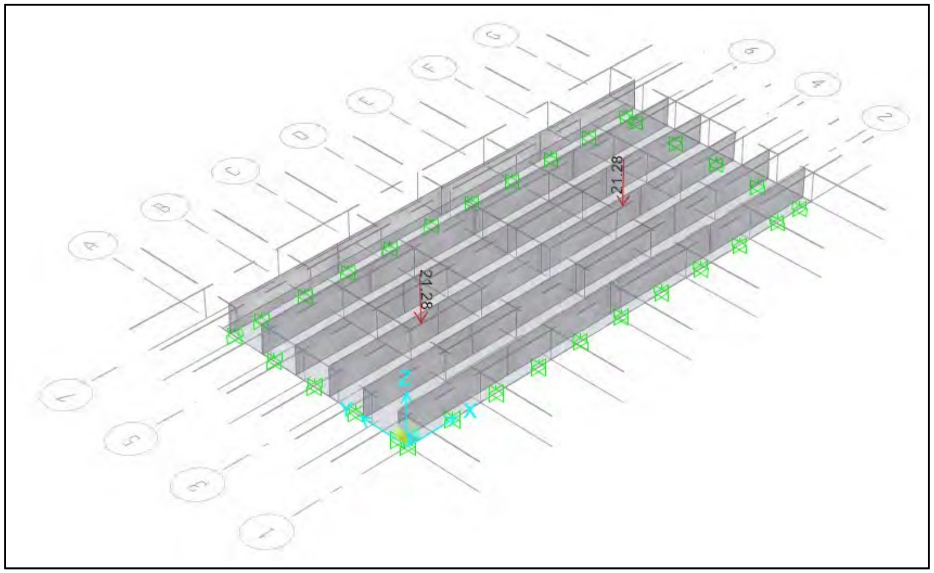


2 Loading Criteria

The Rumble Strip Box loads are self-weight (D) and superimposed traffic loads (LL+IM) equivalent to the (1.33*16 K) calculated separately as 21.28 k/tire area applied 6 ft apart per the AASHTO HL-93 axle width in two cases.



LL+IM Case 1 (21.28k Tire Area at GL-A and GL-D (midpoint))



LL+IM Case 2 (21.28k Tire Area at GL-C and GL-E (midpoint))

The following Load Combinations were ran:

Strength I:	1.25(D) + 1.75(LL+IM case 1)
Service I:	(D) + (LL+IM case 1)
Strength II:	1.25(D) + 1.75(LL+IM case 2)
Service II:	(D) + (LL+IM case 2)
ACI 318 LC1:	1.4(D)

3 Analysis Results

3.1 Service Level Soil Pressures

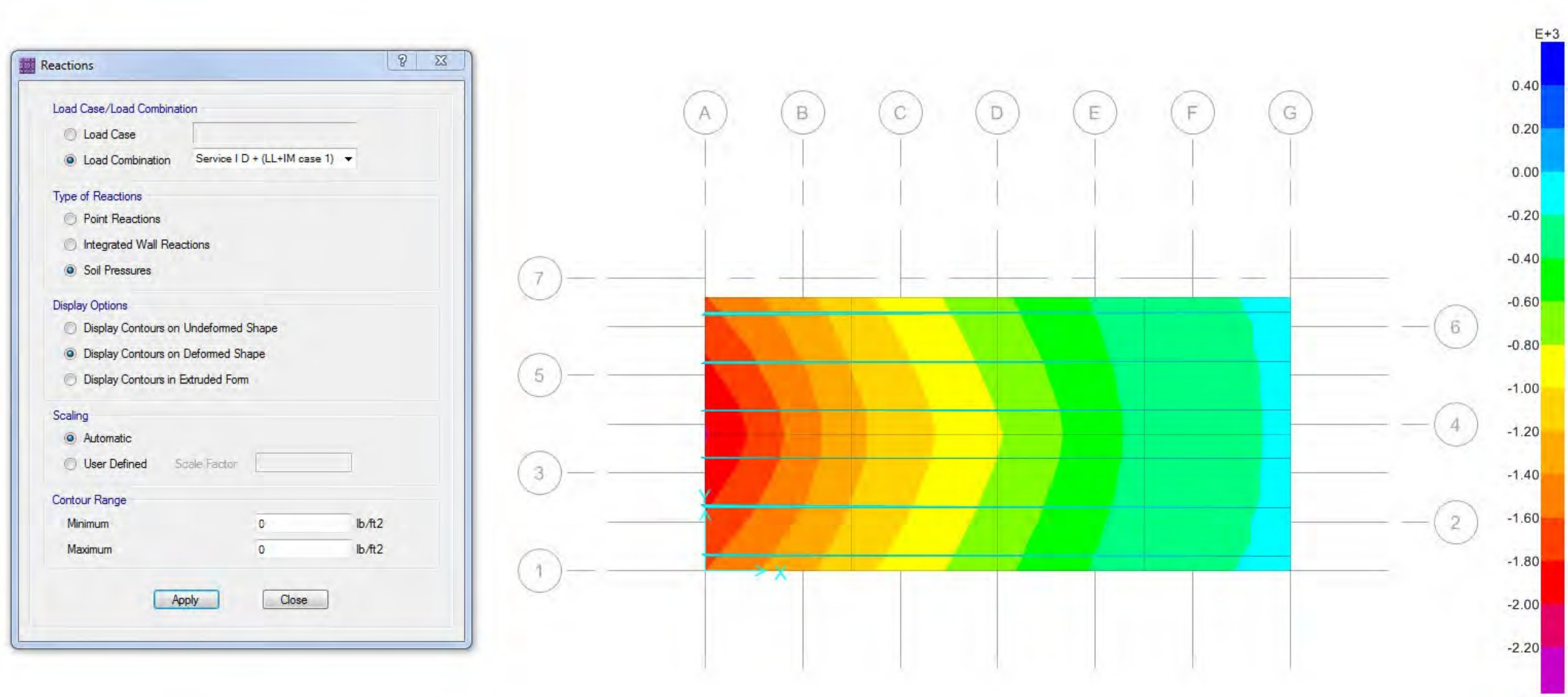


Figure 1: Service I Soil Bearing Pressures (MAX = 2022.47 PSF)

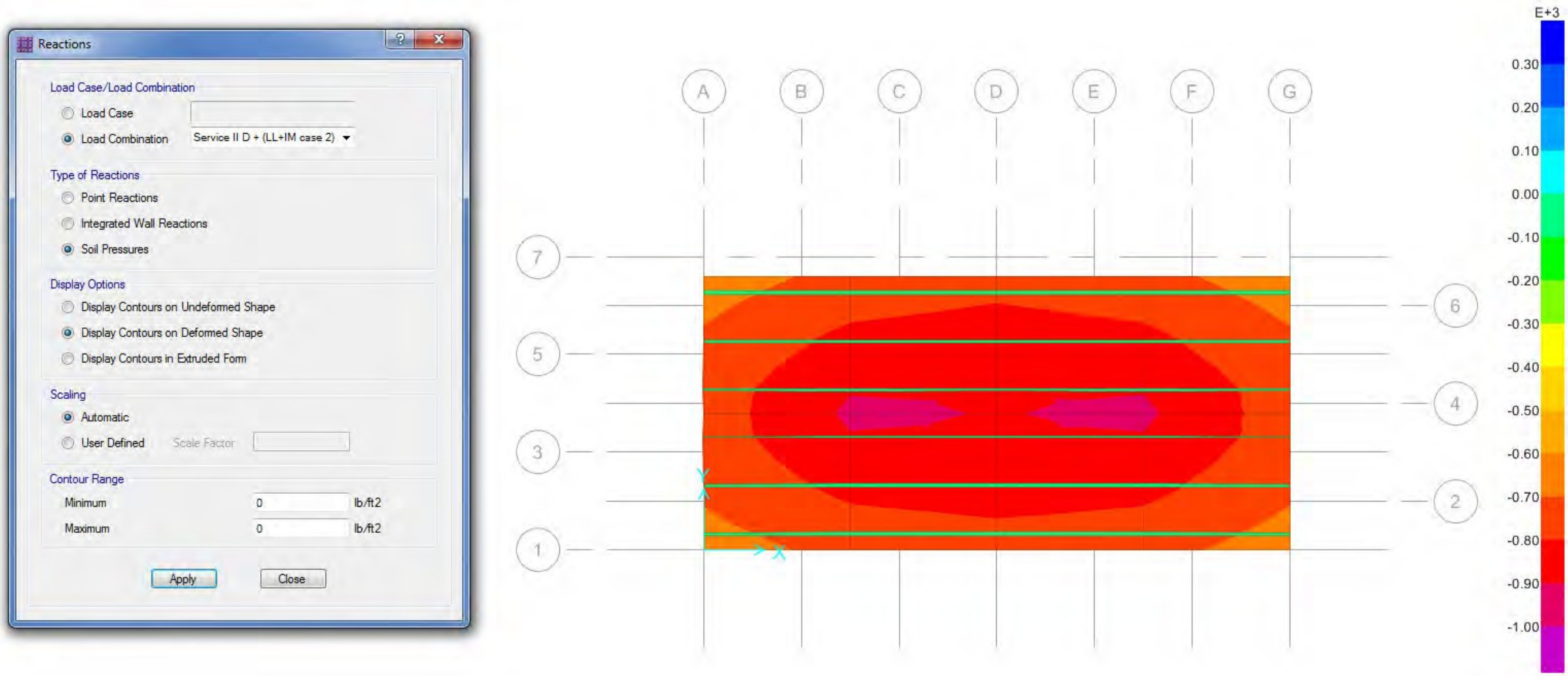


Figure 2: Service II Soil Bearing Pressures (MAX = 917.53 PSF)

3.2 Deflection

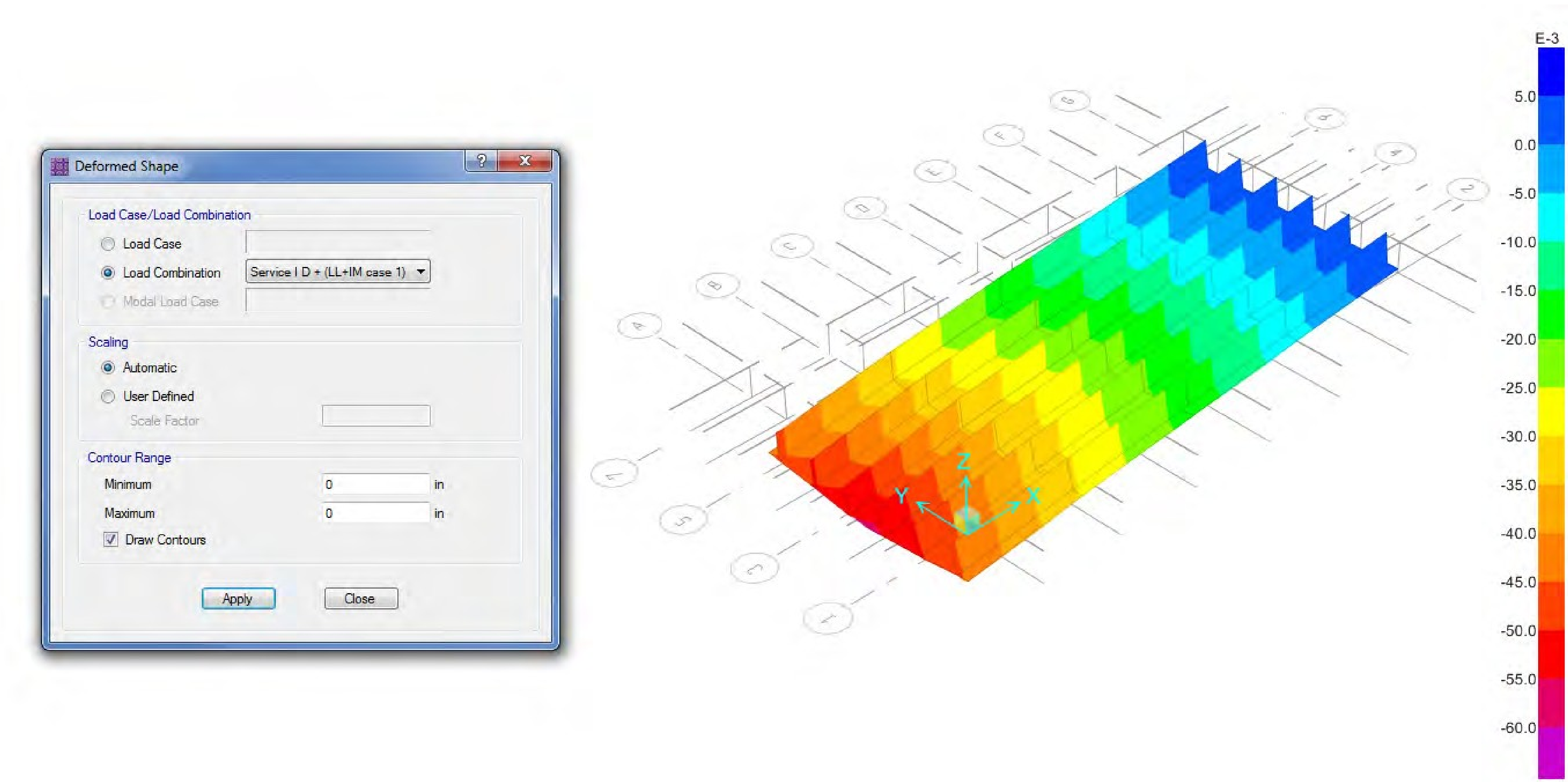


Figure 3: Service I Deflection (MAX = -0.05618 in)

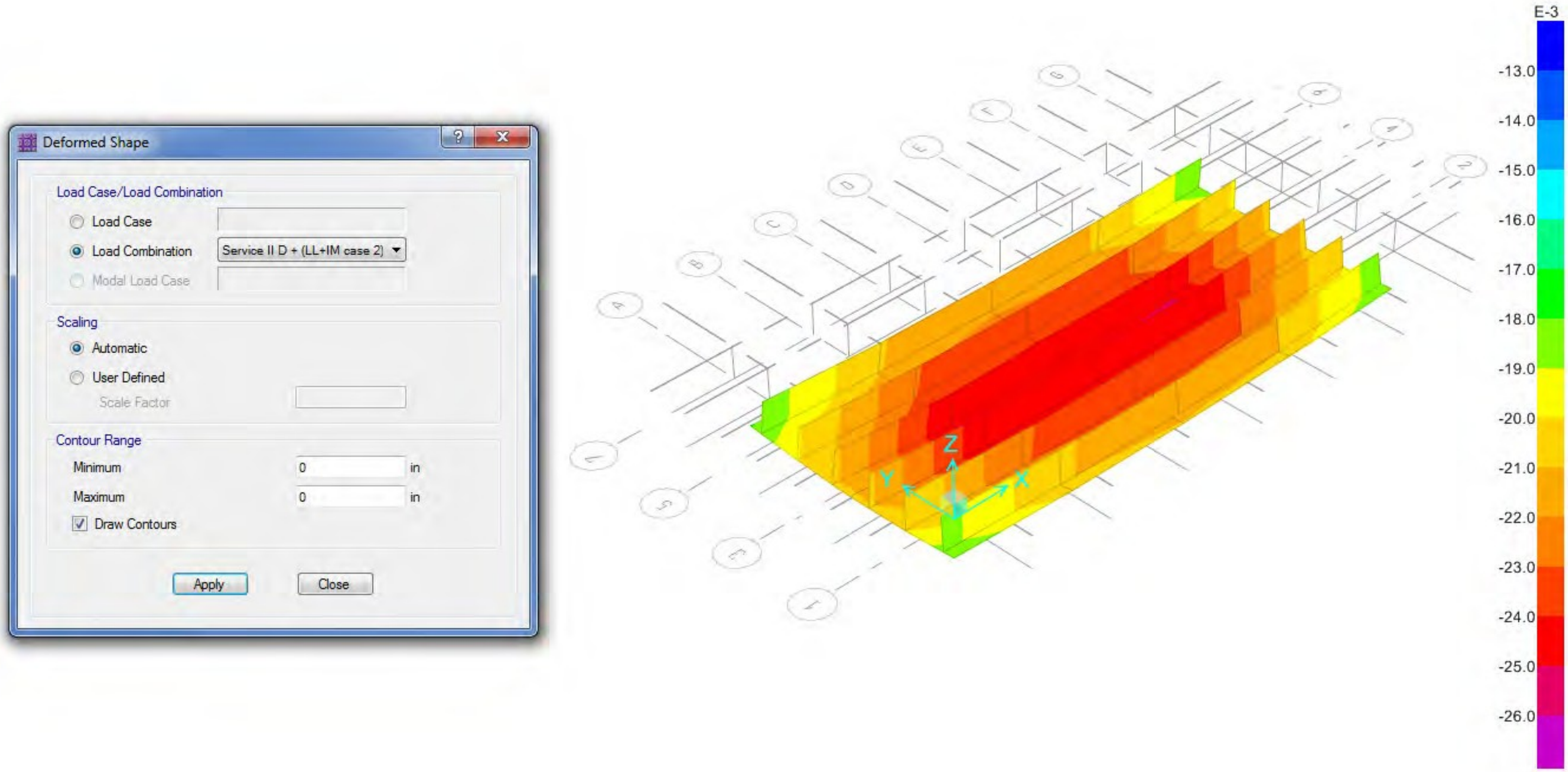


Figure 4: Service II Deflection (MAX=-0.025487")

4 Flexural Demands – Stress Distribution Maps

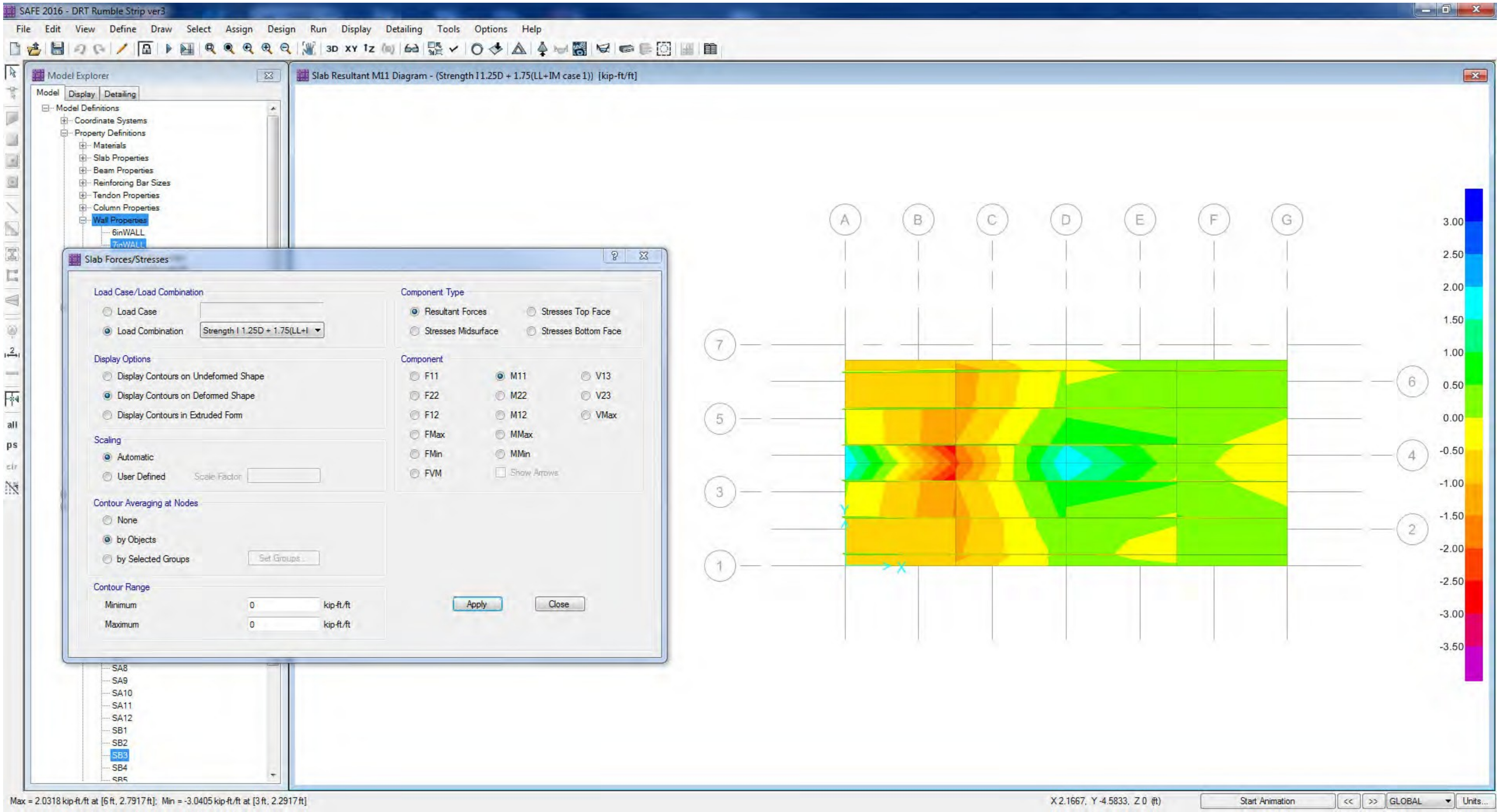


Figure 5: Strength I Flexural Demand (M11 Bending around global Y-Axis), (kip*ft/ft)

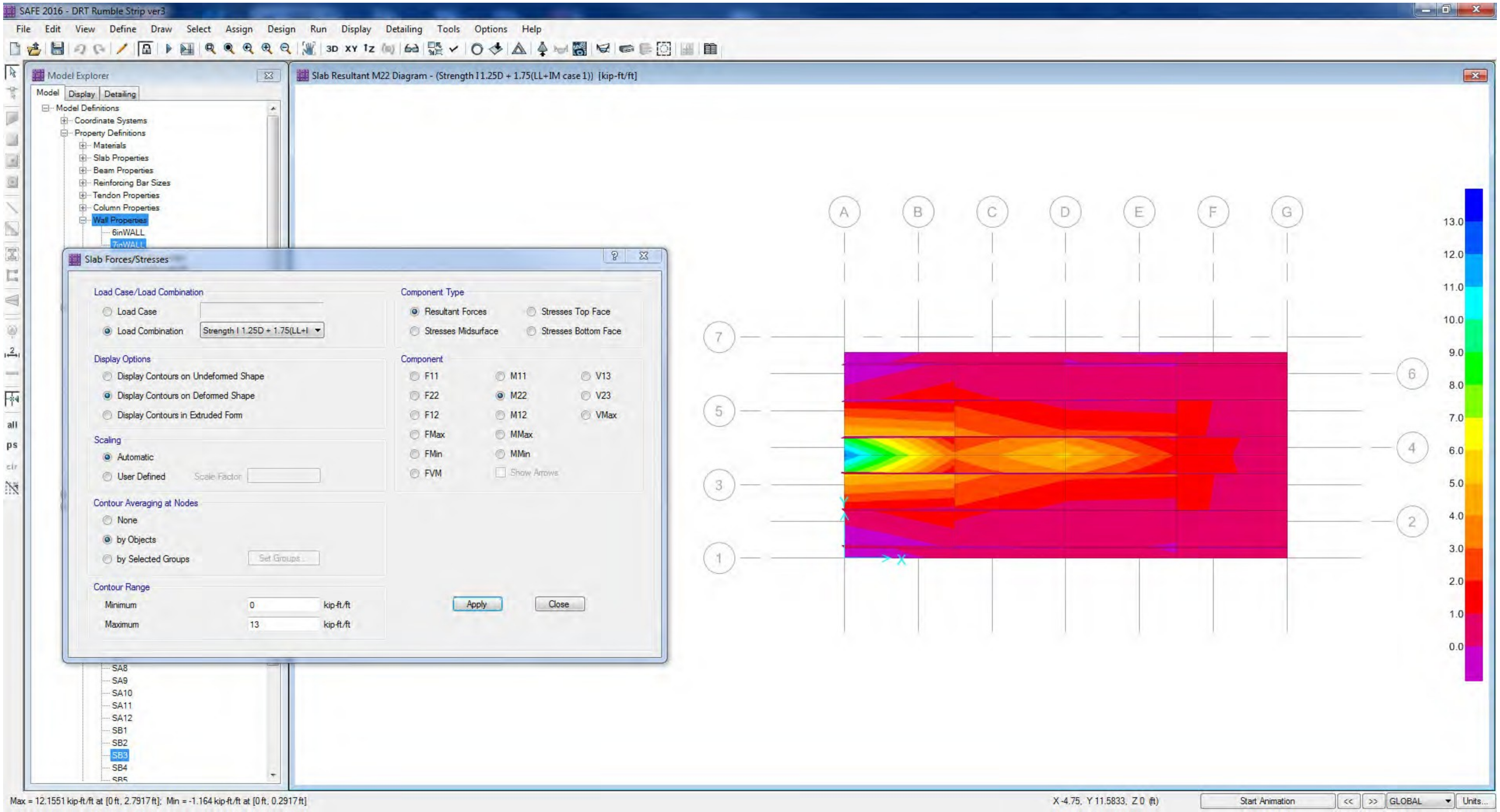


Figure 6: Strength I Flexural Demand (M22 Bending around global X-Axis), (kip*ft/ft)

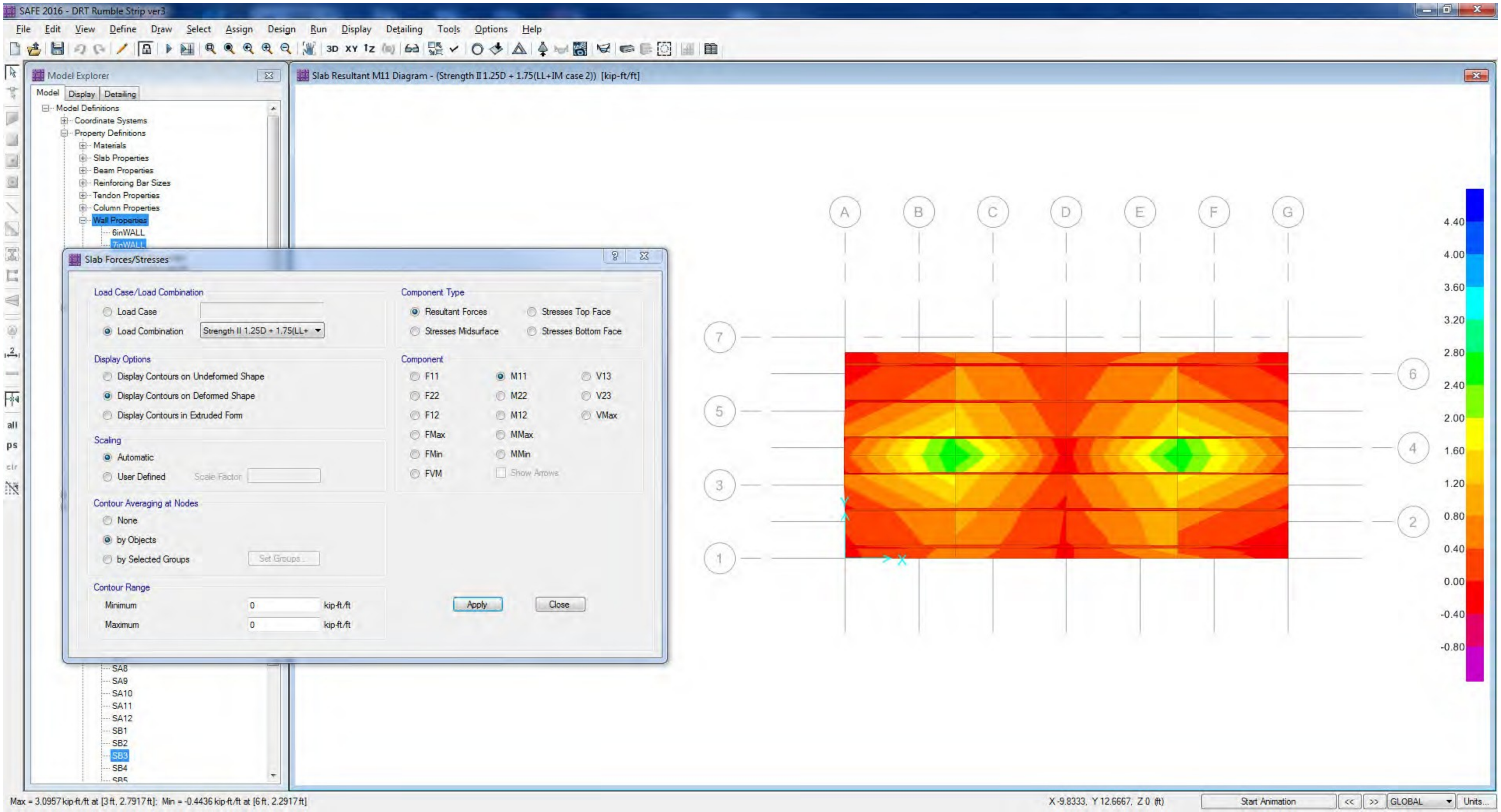


Figure 7: Strength II Flexural Demand (M11 Bending around global Y-Axis), (kip*ft/ft)

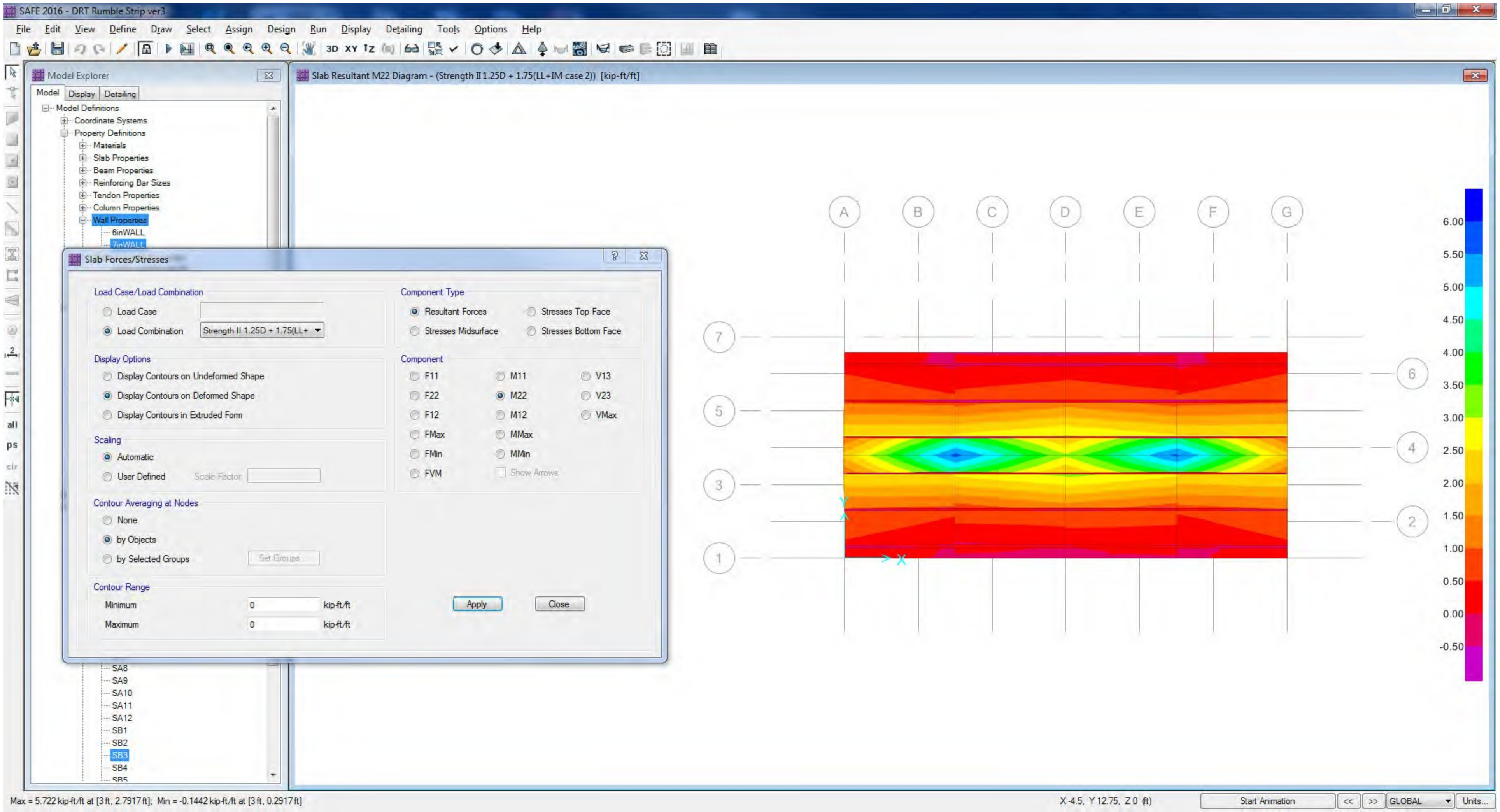


Figure 8: Strength II Flexural Demand (M22 Bending around global X-Axis), (kip*ft/ft)

5 Shear Demand – Stress Distribution Maps

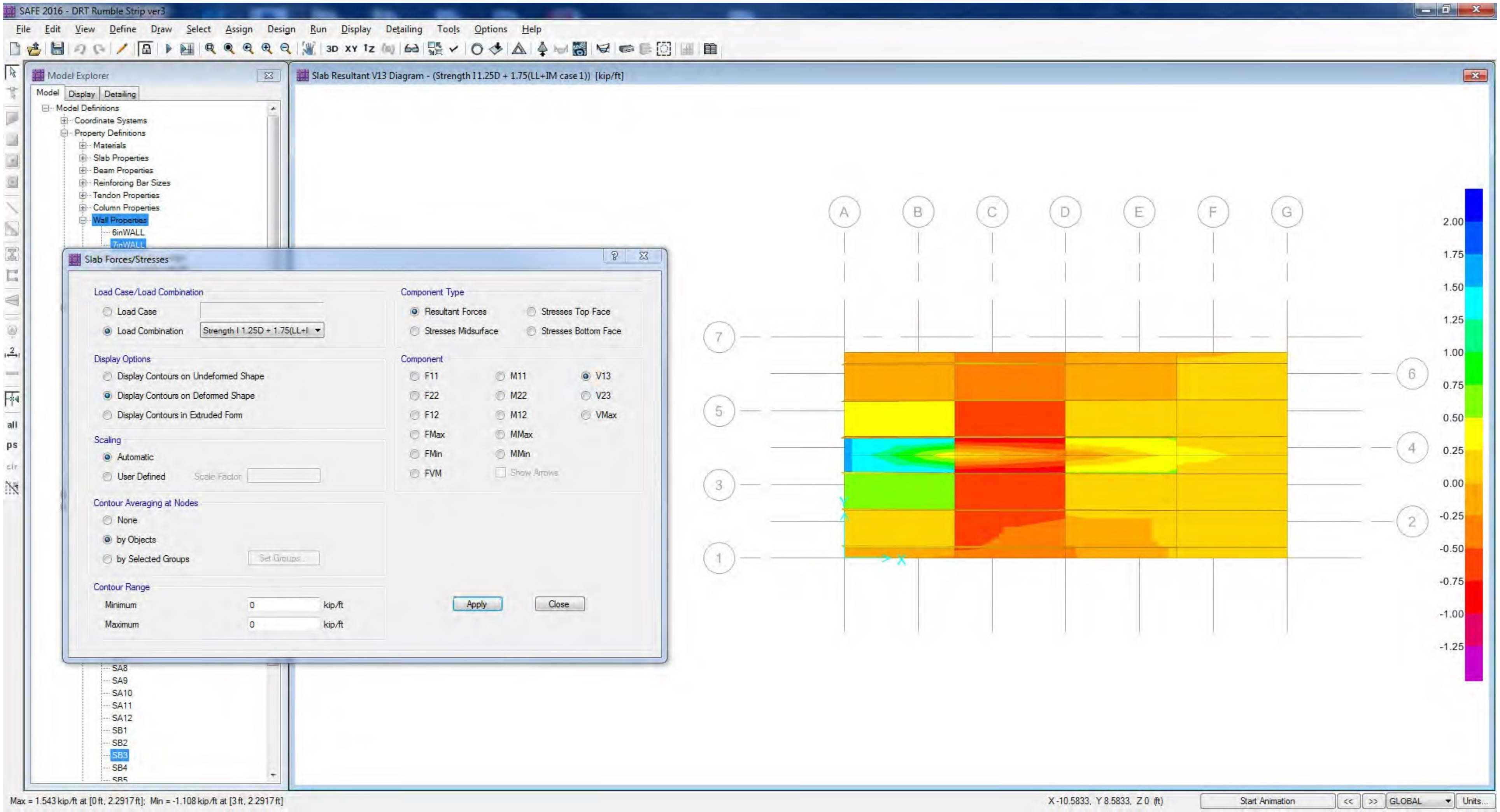


Figure 9: Strength I Shear Demand (V13 Shear plane parallel to global Y-Axis), (kip/ft)

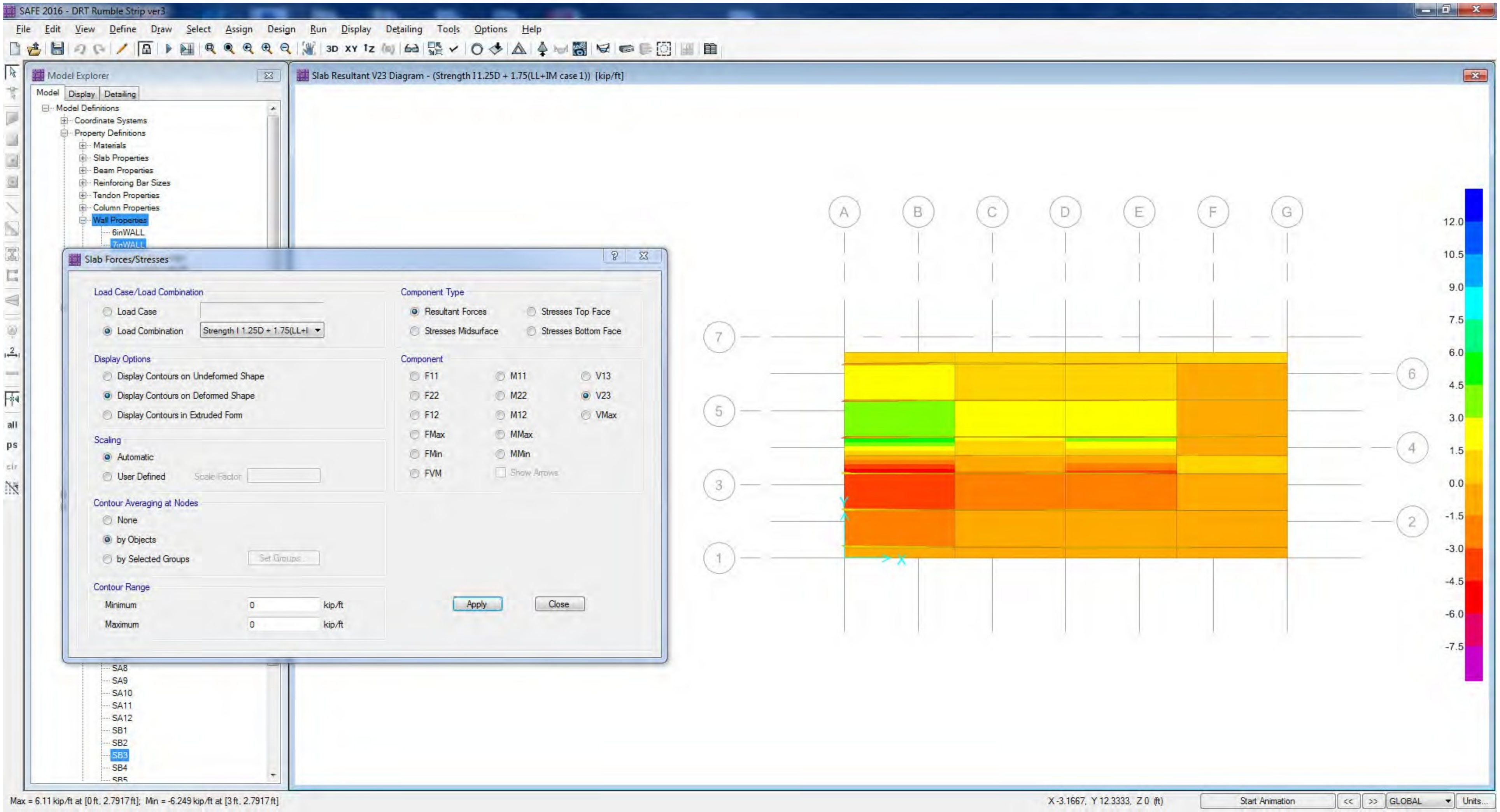


Figure 10: Strength I Shear Demand (V23 Shear plane parallel to global X-Axis), (kip/ft)

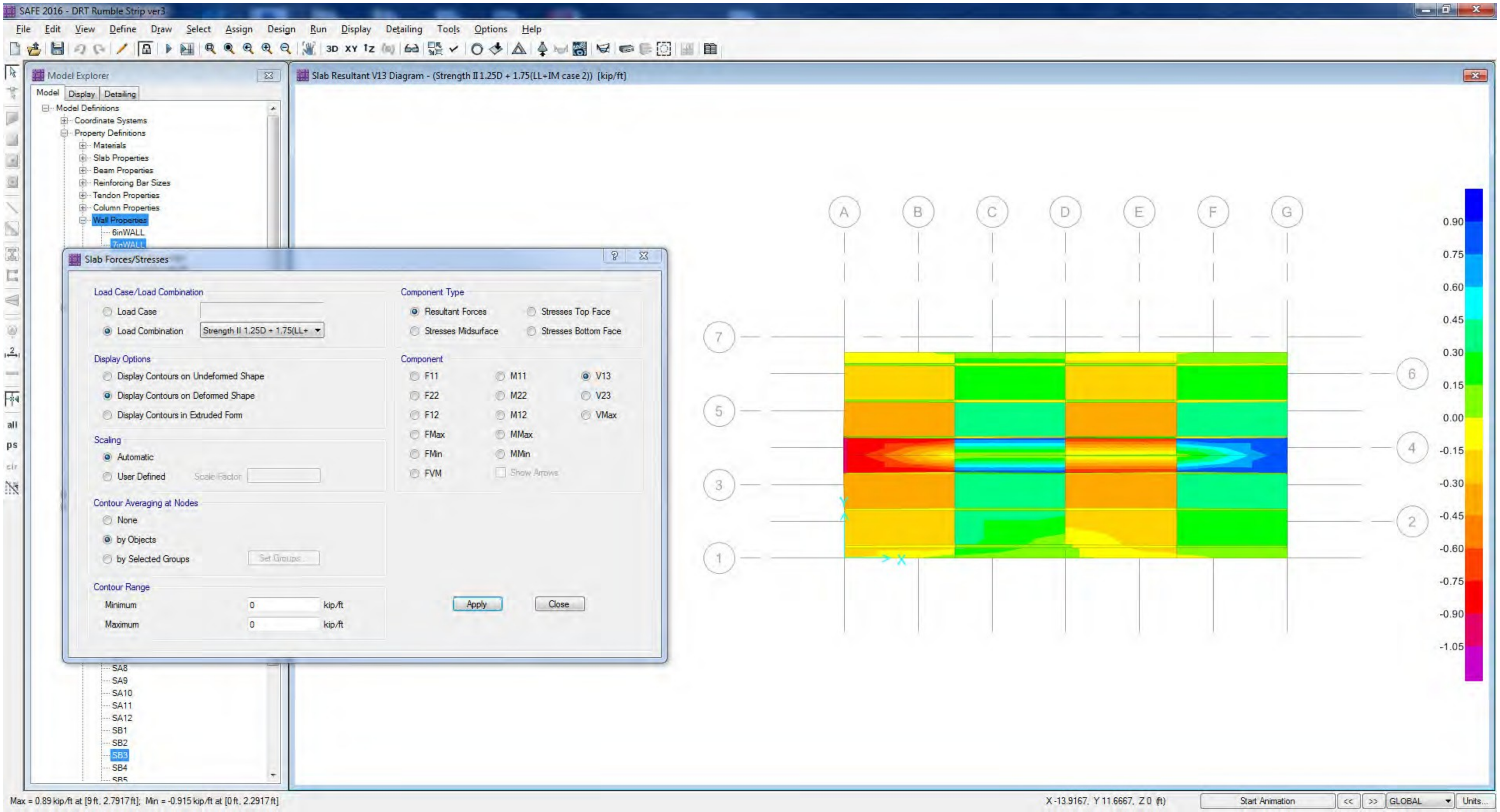


Figure 11: Strength II Shear Demand (V13 Shear plane parallel to global Y-Axis), (kip/ft)

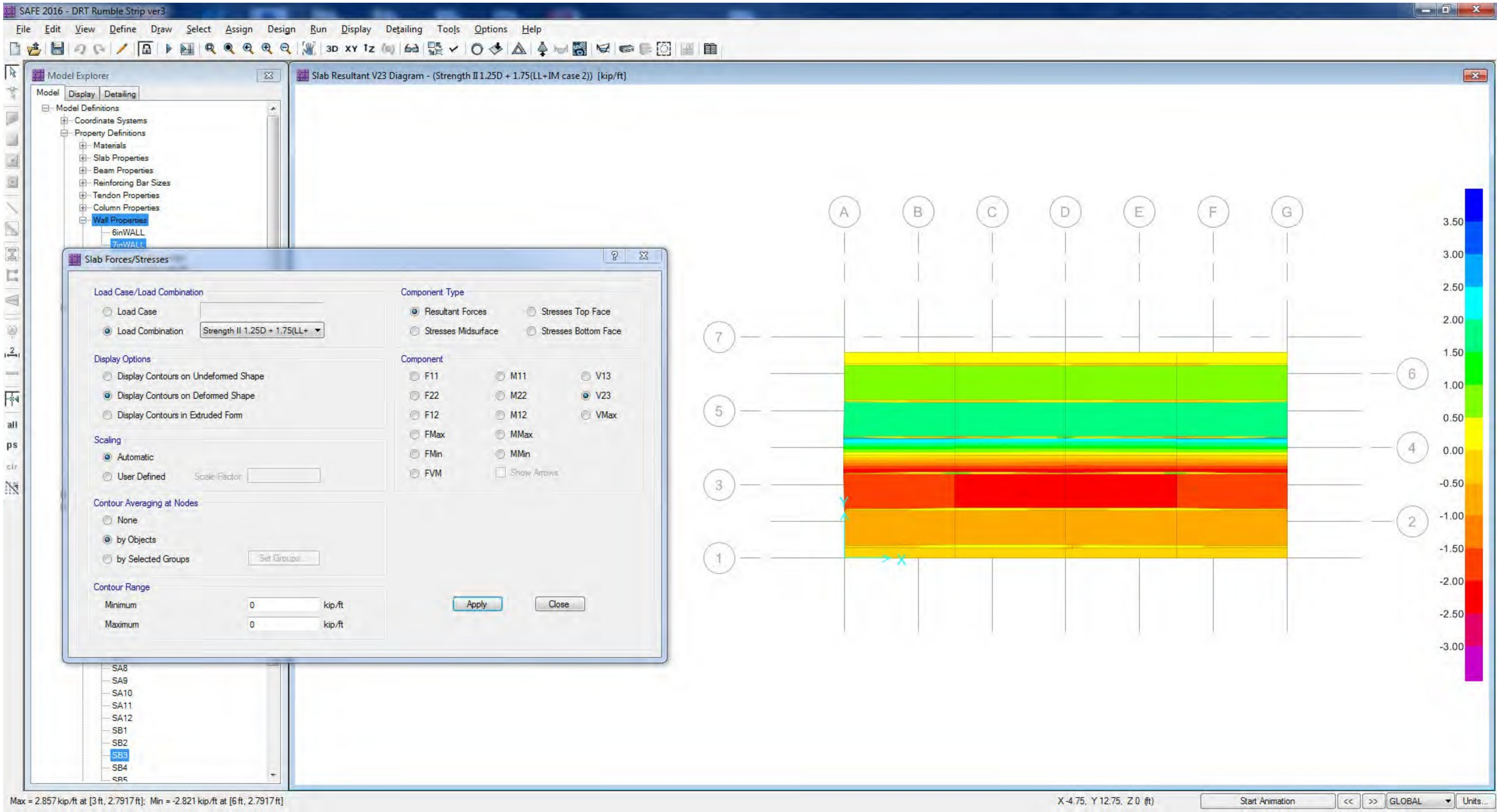


Figure 12: Strength II Shear Demand (V23 Shear plane parallel to global X-Axis), (kip/ft)

6 Design Strips – Flexure & Shear

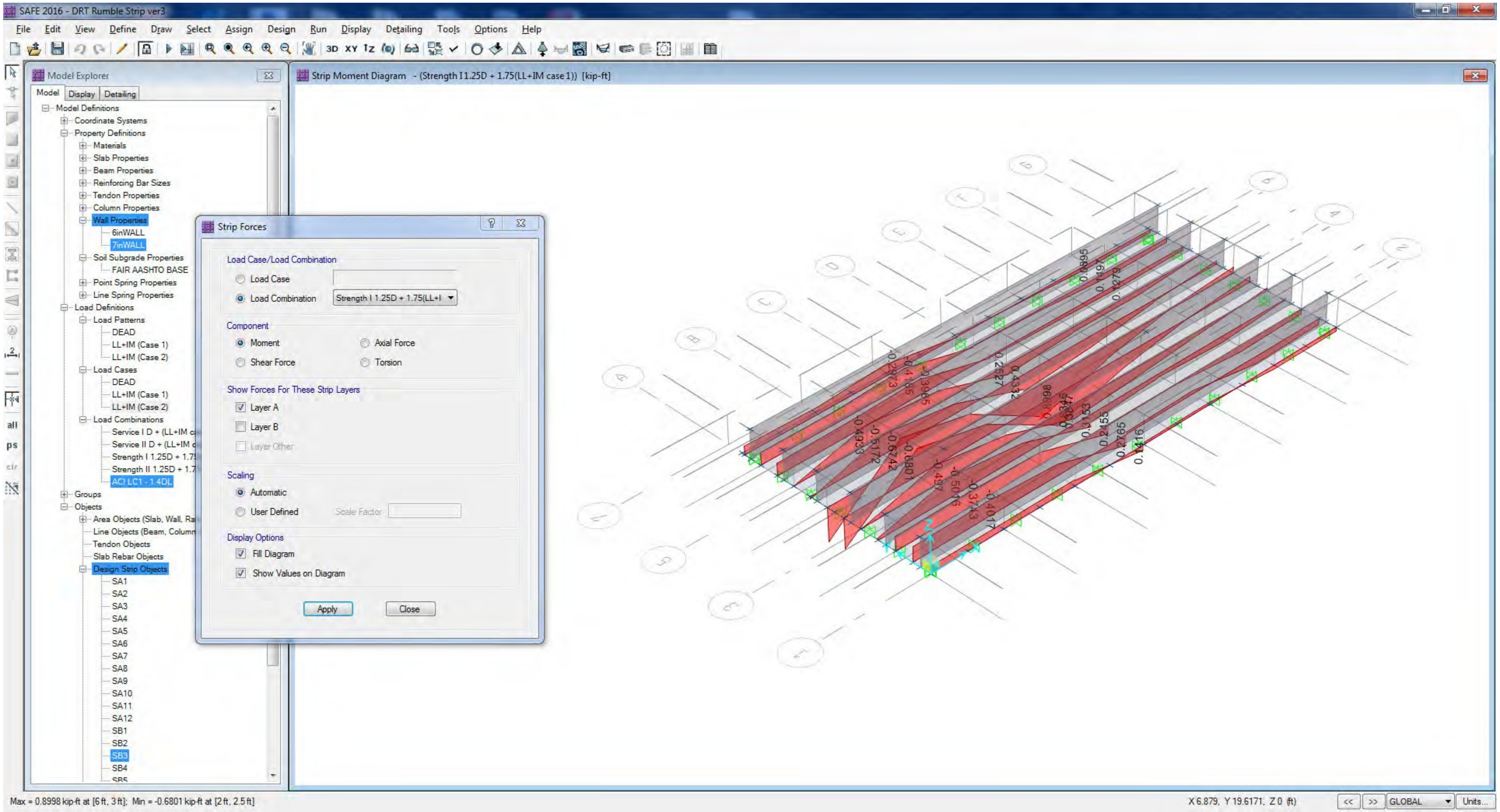


Figure 13: Strength I Design Strip MOMENT along X-Axis (Max 0.8988 k*ft | Min -0.6801 k*ft)

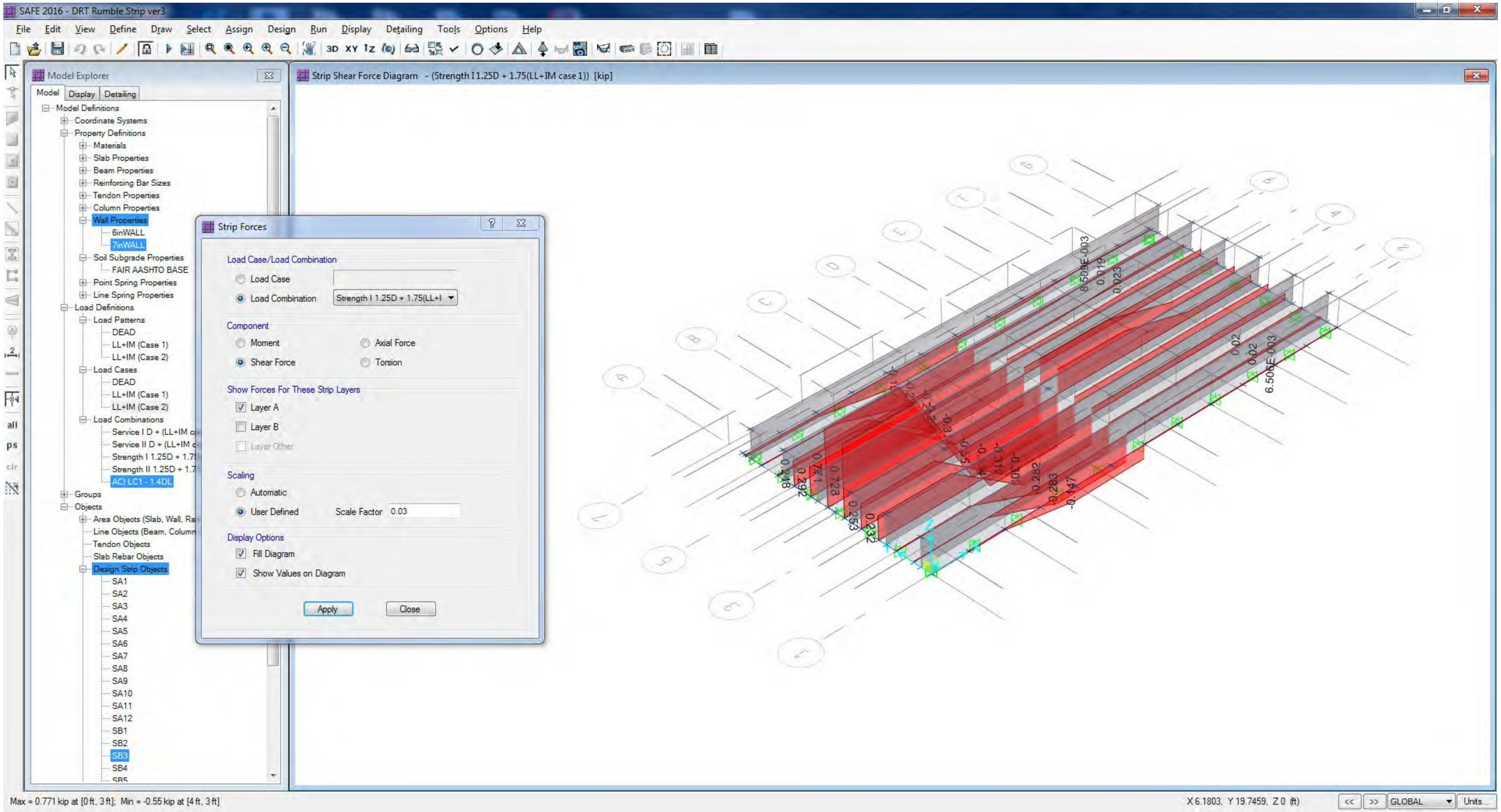


Figure 14: Strength I Design Strip SHEAR along X-Axis (Max 0.771 k | Min -0.55 k)

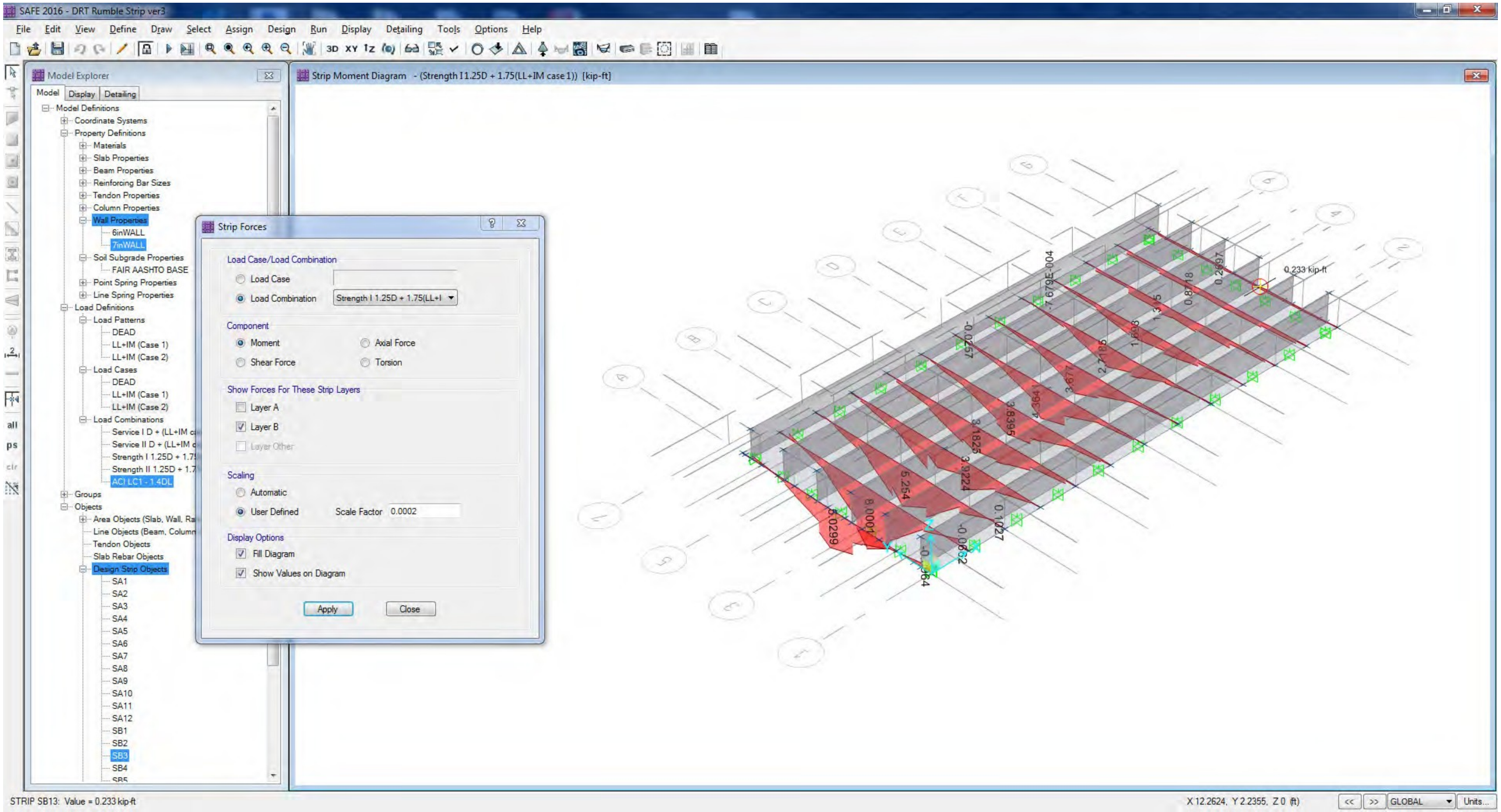


Figure 15: Strength I Design Strip MOMENT along Y-Axis (Max 8.0001 k*ft | Min -0.7301 k*ft)

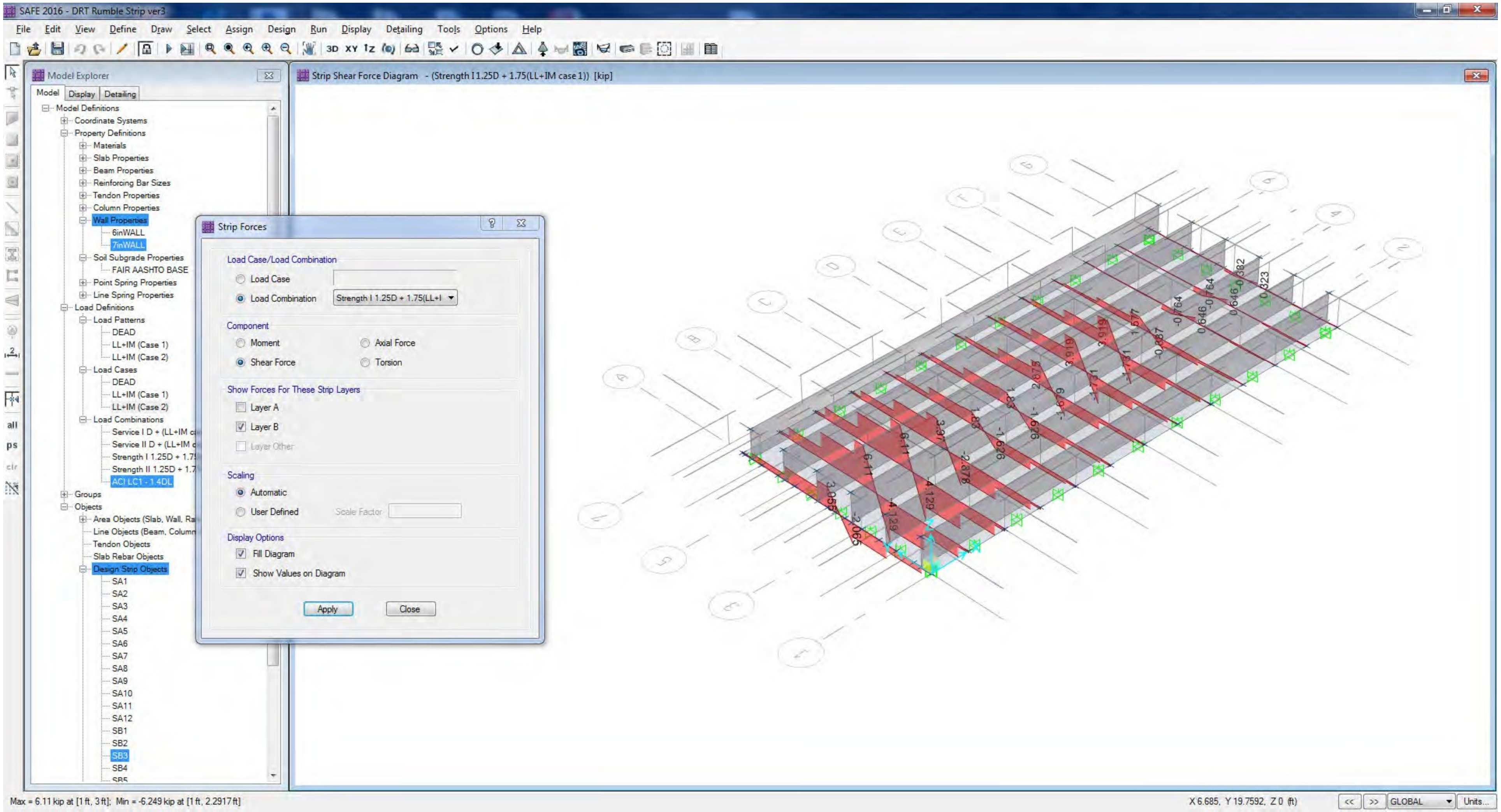


Figure 16: Strength I Design Strip SHEAR along Y-Axis (Max 6.11 k | Min -6.249 k)

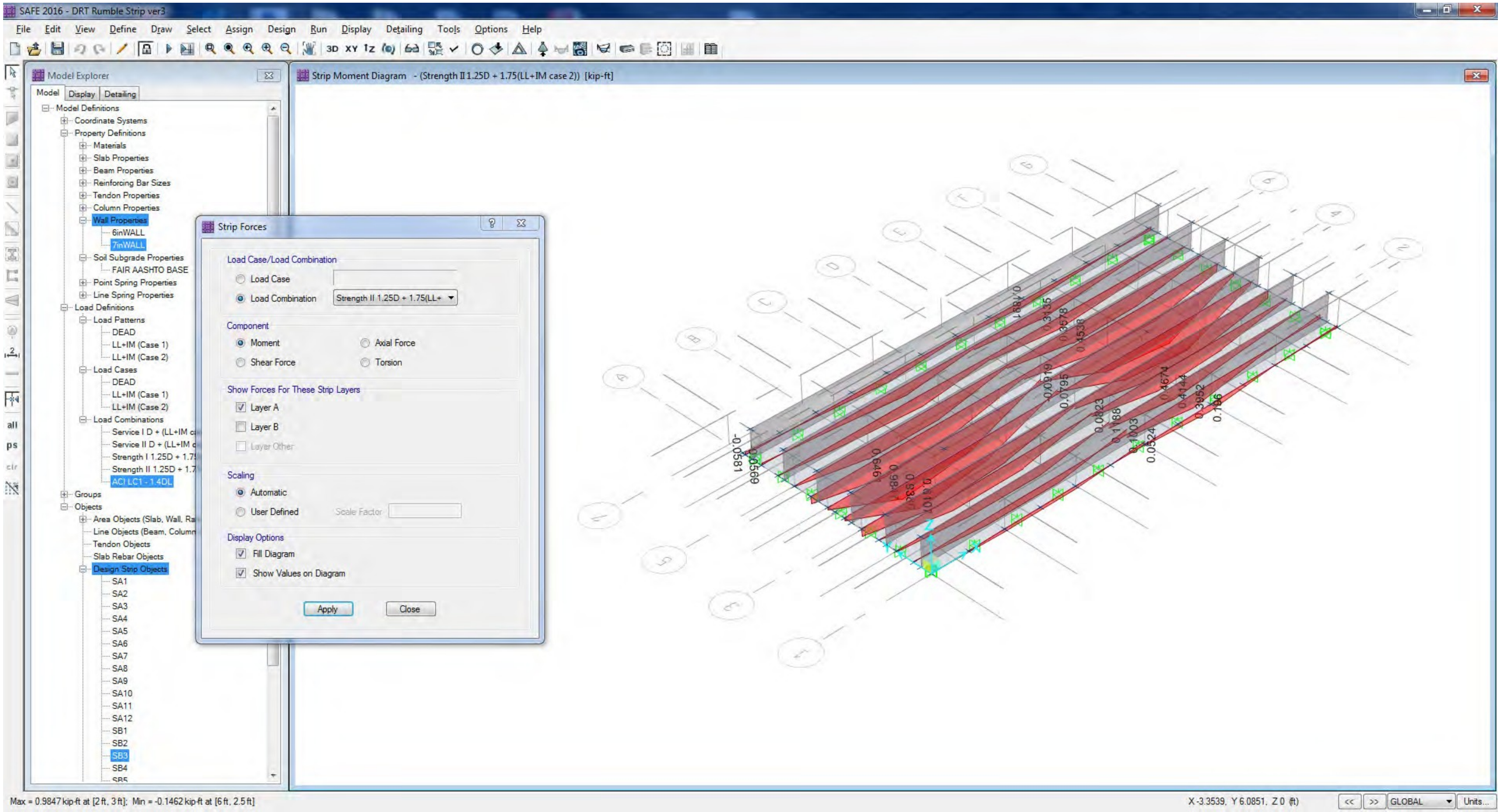


Figure 17: Strength II Design Strip MOMENT along X-Axis (Max 0.9847 k*ft | Min -0.1462 k*ft)

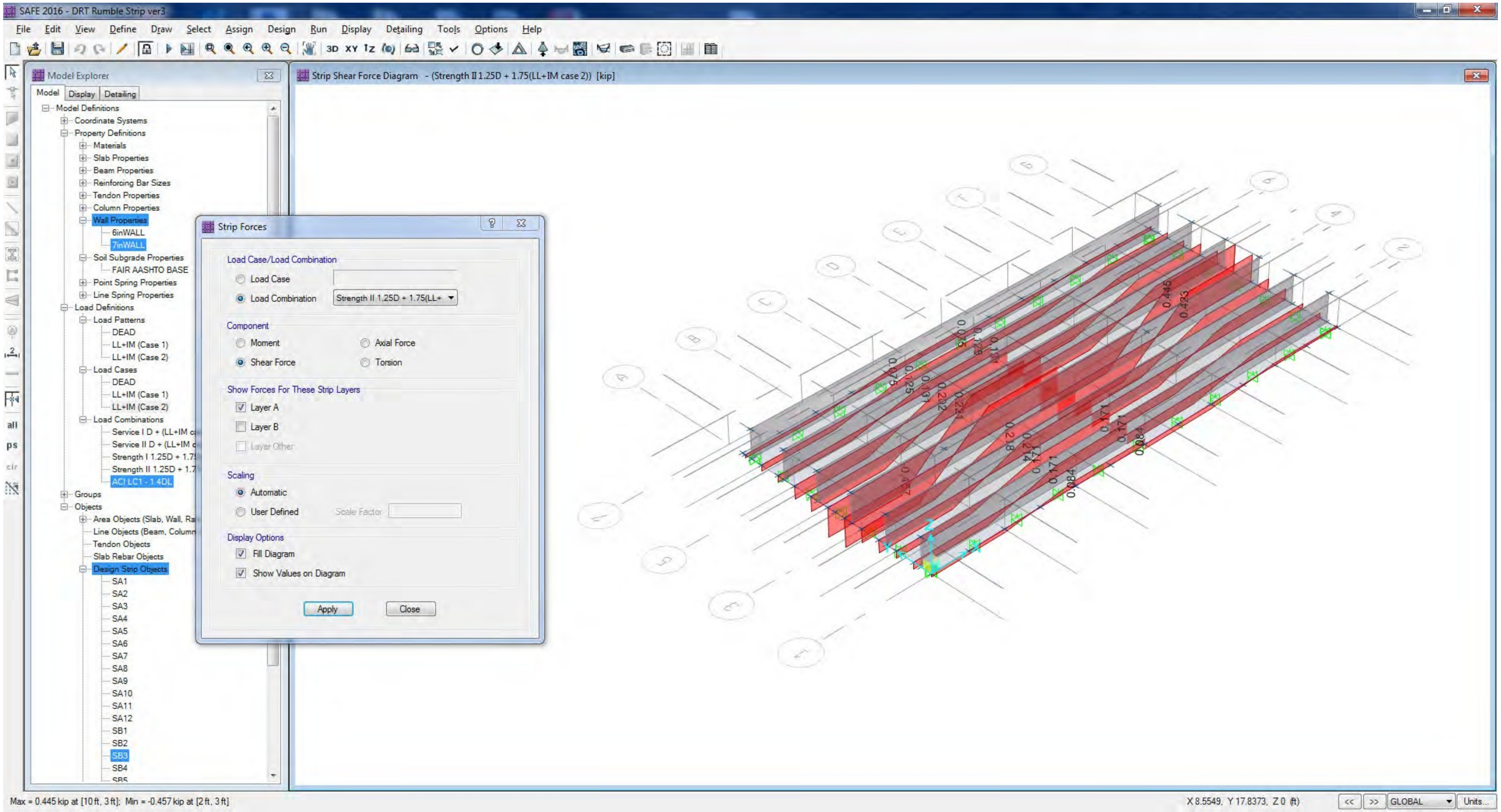


Figure 18: Strength II 6 in Design Strip SHEAR along X-Axis (Max 0.445 k | Min -0.457 k)

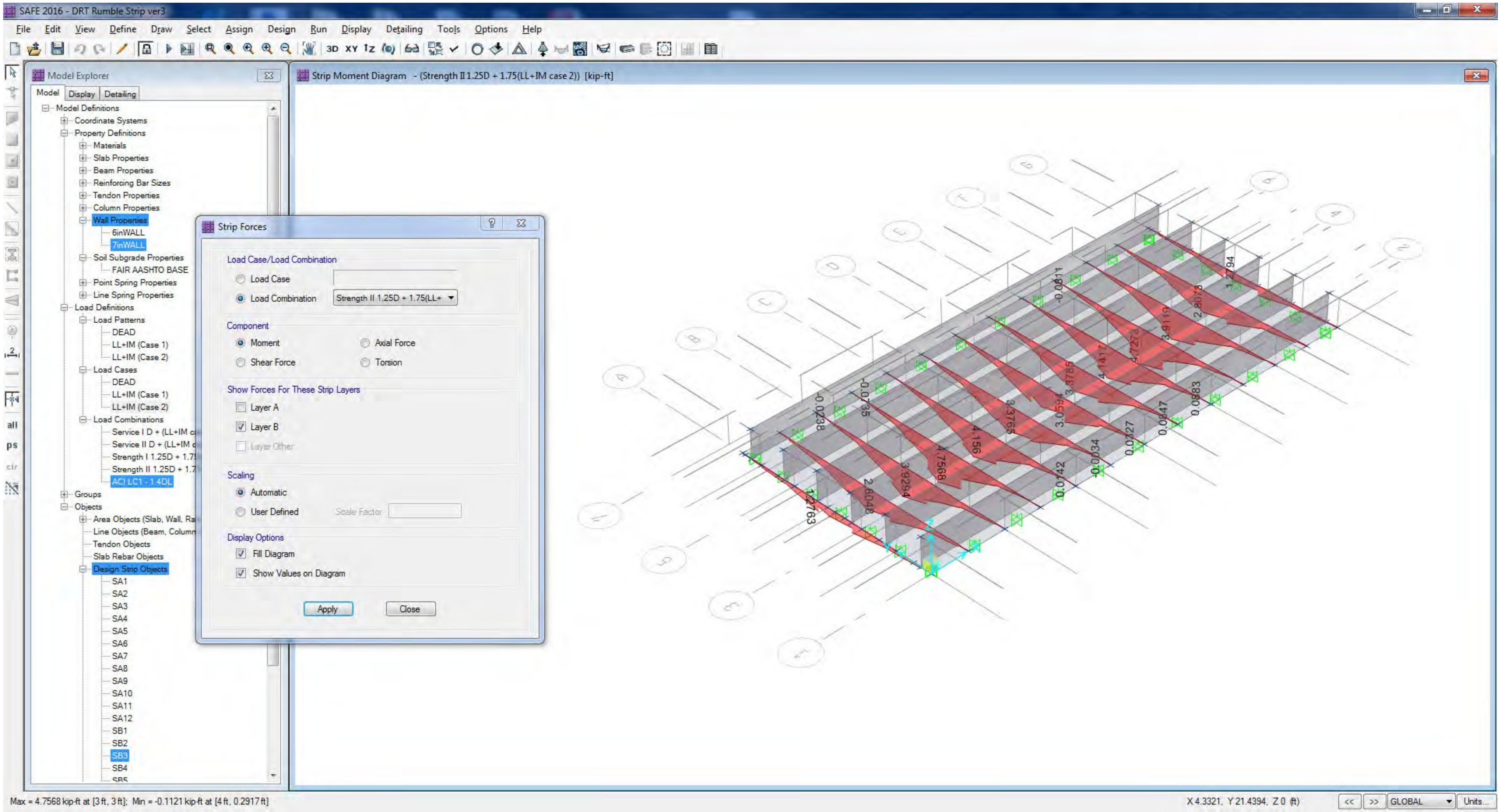


Figure 19: Strength II Design Strip MOMENT along Y-Axis (Max 4.7568 k*ft | Min -0.1121 k*ft)

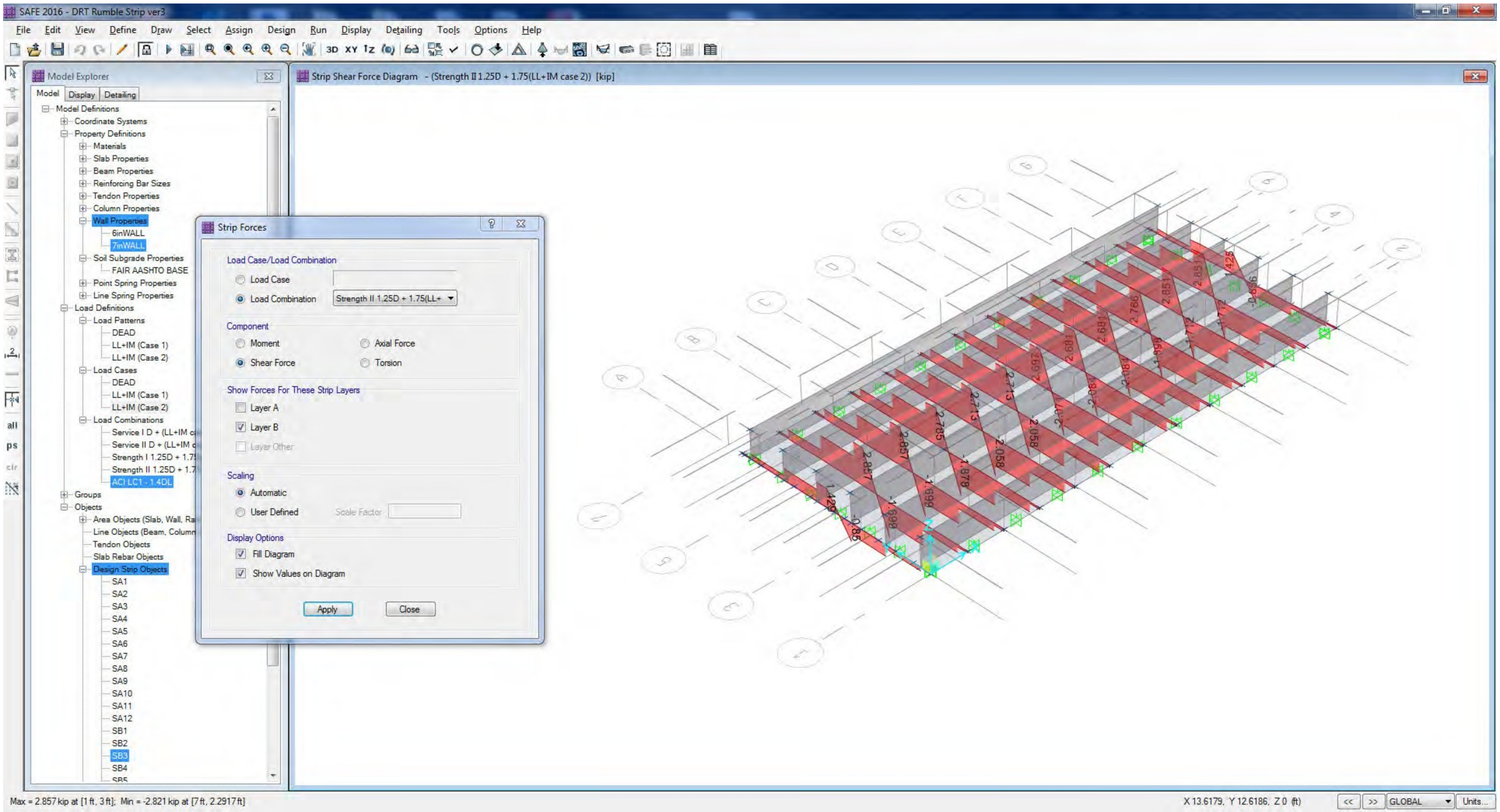


Figure 20: Strength II Design Strip SHEAR along Y-Axis (Max 2.857 k | Min -2.821 k)

Supporting Full Height Concrete Slab

Reference

Below each rumble strip mechanism there is a supporting concrete slab that spans the width of the box and extends the defined "h" depth below the mechanisms. The dimensions and criteria are defined here:

preliminary design

$b := 12\ in$ width of concrete beam section

$h := 8\ in$ height of concrete beam section

$cvr := 2\ in$ required clear cover

$f_{y.bar} := 60\ ksi$ concrete reinf. steel yield strength (Gr. 60 reinforcement)

$f'_c := 4500\ psi$ concrete compressive strength

$E_s := 29000\ ksi$ modulus of Elasticity for Steel

$\gamma_{conc} := 150\ pcf$ unit Weight of Concrete

Choose a reinforcement size to check:

$A_{s.bar} := 5$ flexural reinforcement bar size

$d_{bar}(A_{s.bar}) = 0.625\ in$ bar diameter

$A_{se}(A_{s.bar}) = 0.31\ in^2$ bar area

$$d_{bar}(x) := \begin{cases} 0.375\ in & \text{if } x=3 \\ 0.500\ in & \text{if } x=4 \\ 0.625\ in & \text{if } x=5 \\ 0.750\ in & \text{if } x=6 \\ 0.875\ in & \text{if } x=7 \\ 1.000\ in & \text{if } x=8 \\ 1.128\ in & \text{if } x=9 \\ 1.270\ in & \text{if } x=10 \\ 1.410\ in & \text{if } x=11 \\ 1.693\ in & \text{if } x=14 \\ 2.257\ in & \text{if } x=18 \end{cases}
\qquad
A_{se}(x) := \begin{cases} 0.11\ in^2 & \text{if } x=3 \\ 0.20\ in^2 & \text{if } x=4 \\ 0.31\ in^2 & \text{if } x=5 \\ 0.44\ in^2 & \text{if } x=6 \\ 0.60\ in^2 & \text{if } x=7 \\ 0.79\ in^2 & \text{if } x=8 \\ 1.00\ in^2 & \text{if } x=9 \\ 1.27\ in^2 & \text{if } x=10 \\ 1.56\ in^2 & \text{if } x=11 \\ 2.25\ in^2 & \text{if } x=14 \\ 4.00\ in^2 & \text{if } x=18 \end{cases}$$

$$d_e := h - cvr - \frac{d_{bar}(A_{s.bar})}{2}$$

$d_e = 5.69\ in$ effective depth

Flexural & Shear Demands:
These are the maximum factored moment (M_u) and shear (V_u) that the beam needs to carry found through a STAAD analysis:

$M_u := 8.001\ kip \cdot ft$ Flexural demand
1.25DL+1.75(LL+IM)
see SAFE Output Figure 15

$V_u := 6.249\ kip$ Shear demand
1.25DL+1.75(LL+IM)
see SAFE Output Figure 16

Flexural Design (Singly Reinforced Option):
The reinforcement chosen needs to provide a nominal flexural strength that exceeds the maximum factored moment (M_u) found through a STAAD analysis:

$\phi_b := 0.90$ strength reduction factor for tension controlled sections — ACI 318-11 9.3.2

$$R_{n.req'd} := \frac{M_u}{\phi_b \cdot b \cdot {d_e}^2} = 275\ psi$$

required coefficient for resistance for strength design

$$m := \frac{f_{y.bar}}{0.85 \cdot f'_c} = 15.69$$

stress ratio

Reference

$$\rho_{req'd} := \frac{1}{m}\left(1-\sqrt{1-\frac{2 \cdot m \cdot R_{n.req'd}}{f_{y.bar}}}\right) = 0.0048$$

required ratio for strength design

$$\rho_{min} := \max\left(\frac{3 \cdot \sqrt{\frac{f'_c}{psi}}}{f_{y.bar}} \cdot psi, \frac{200}{f_{y.bar}} \cdot psi\right) = 0.0034$$

$$A_{s.req'd} := \max\left(\rho_{req'd}, \rho_{min}\right) \cdot b \cdot d_e$$

required flexural reinforcement in tension

$$A_{s.req'd} = 0.325\ in^2$$

$$\#ofA_{s.bar} := 3$$

number of bars to use for tensile reinforcement

$$A_{se}\left(A_{s.bar}\right) = 0.31\ in^2$$

$$A_s := \#ofA_{s.bar} \cdot A_{se}\left(A_{s.bar}\right) = 0.930\ in^2$$

provided steel reinforcement area

$$\beta_1 := \begin{cases} \text{if } f'_c \le 4000\ psi \\ \quad 0.85 \\ \text{if } f'_c > 4000\ psi \\ \quad \max\left(0.85 - 0.05 \cdot \left(\frac{f'_c - 4000\ psi}{1000\ psi}\right), 0.65\right) \end{cases} = 0.83$$

ratio of the depth of rectangular stress distribution to the depth of the neutral axis

ACI 318-11 10.2.7.3

$$\rho_{prov} := \frac{A_s}{b \cdot d_e} = 0.0136$$

$$\varepsilon_y := \frac{f_{y.bar}}{E_s} = 0.0021$$

tension yielding strain limit for reinforcement steel

$$\rho_b := \frac{0.85 \cdot f'_c}{f_{y.bar}} \cdot \beta_1 \cdot \left(\frac{0.003 \cdot E_s}{0.003 \cdot E_s + f_{y.bar}}\right) = 0.0311$$

balanced strain condition

$$\rho_{max} := \frac{0.003 + \varepsilon_y}{0.007} \cdot \rho_b = 0.0225$$

maximum reinforcement ratio at ε_t=0.004 which is the lowest permitted steel strain at ult strength of flexural members

$$check.\rho_{max} = \text{"OK"}$$

$$check.\rho_{max} := \text{if } \rho_{max} > \rho_{prov} \;\; \text{"OK"} \;\; \text{else} \;\; \text{"NOT OK"}$$

Check the design as follows:

$$T_b := A_s \cdot f_{y.bar} = 55.8\ kip$$

Tensile capacity of reinforcement steel

$$a_b := \frac{T_b}{0.85 \cdot f'_c \cdot b} = 1.22\ in$$

Whitney Equivalent rectangular block depth

$$M_n := T_b \cdot \left(d_e - \frac{a_b}{2}\right) = 23.62\ kip \cdot ft$$

nominal flexural strength

$$x_b := \frac{a_b}{\beta_1} = 1.47\ in$$

the neutral axis is located x_b from the point of maximum compression

Reference

$\varepsilon_t := \left(\frac{d_e - x_b}{x_b}\right) \cdot 0.003 = 0.0086$ net strain in the tension steel

$$check.\varepsilon_s := \begin{cases} \text{"Compression Controlled"} & \text{if } \varepsilon_t \le 0.002 \\ \text{"Transion Range"} & \text{if } 0.002 < \varepsilon_t < 0.005 \\ \text{"Tension Controlled"} & \text{if } \varepsilon_t \ge 0.005 \end{cases}$$

$$\phi_{b.adj} := \begin{cases} 0.65 & \text{if } \varepsilon_t \le 0.002 \\ 0.65 + (\varepsilon_t - 0.002) \cdot \left(\frac{250}{3}\right) & \text{if } 0.002 < \varepsilon_t < 0.005 \\ 0.90 & \text{if } \varepsilon_t \ge 0.005 \end{cases}$$

ACI 318-11
9.3.2
Fig. R9.3.2

$check.\varepsilon_s = \text{"Tension Controlled"}$

$\phi_{b.adj} = 0.90$

$\phi M_n := \phi_{b.adj} \cdot M_n = 21.26\ kip \cdot ft$

$\phi M_n = 21.26\ kip \cdot ft$

$$check.\phi M_n := \begin{cases} \text{"OK"} & \text{if } \phi M_n > M_u \\ \text{"NOT OK"} & \text{else} \end{cases}$$

$M_u = 8\ kip \cdot ft$

$check.\phi M_n = \text{"OK"}$

Shear Capacity Check:
The concrete section nominal shear strength will be checked if it exceeds exceeds the maximum factored shear (V_u) found through the SAFE analysis:

$\phi_v := 0.75$ ACI 318-11 9.3.2.3

$\lambda_{conc} := 1.0$ ACI 318-11 9.6.1

$V_c := 2 \cdot \lambda_{conc} \cdot \sqrt{\frac{f'_c}{psi}} \cdot \frac{b}{in} \cdot \frac{d_e}{in} \cdot lbf = 9.16\ kip$ for members subject to shear and flexure only

ACI 318-11
11.2.1.1
(11-3)

$\phi V_c := \phi_v \cdot V_c = 6.868\ kip$

$$check.\phi V_c := \begin{cases} \text{"OK"} & \text{if } \phi V_c > V_u \\ \text{"NOT OK"} & \text{else} \end{cases}$$

$V_u = 6.25\ kip$

$check.\phi V_c = \text{"OK"}$

Full Height Concrete Beam

Reference

In between each rumble strip mechanism there is 7" solid concrete that spans the length of the box and extends the full depth of the box. The dimensions and criteria are defined here:

preliminary design

$b := 7\ in$ — width of concrete beam section

$h := 17\ in$ — height of concrete beam section (8" slab + 9" stem wall)

$cvr := 2\ in$ — required clear cover

$f_{y.bar} := 60\ ksi$ — concrete reinf. steel yield strength (Gr. 60 reinforcement)

$f'_c := 4500\ psi$ — concrete compressive strength

$E_s := 29000\ ksi$ — modulus of Elasticity for Steel

$\gamma_{conc} := 150\ pcf$ — unit weight of concrete

Choose a reinforcement size to check:

$Ties_{bar} := 5$ — Size of Ties or Perp Bars if present and affects effective depth (d_e)
input 0 if there is no bar or ties reducing d_e

$A_{s.bar} := 6$ — define flexural reinforcement bar size

$d_{bar}(A_{s.bar}) = 0.75\ in$ — bar diameter

$A_{se}(A_{s.bar}) = 0.44\ in^2$ — bar area

$$d_{bar}(x) := \begin{cases} \text{if } x=0: & 0\ in \\ \text{if } x=3: & 0.375\ in \\ \text{if } x=4: & 0.500\ in \\ \text{if } x=5: & 0.625\ in \\ \text{if } x=6: & 0.750\ in \\ \text{if } x=7: & 0.875\ in \\ \text{if } x=8: & 1.000\ in \\ \text{if } x=9: & 1.128\ in \\ \text{if } x=10: & 1.270\ in \\ \text{if } x=11: & 1.410\ in \\ \text{if } x=14: & 1.693\ in \\ \text{if } x=18: & 2.257\ in \end{cases}$$

$$A_{se}(x) := \begin{cases} \text{if } x=3: & 0.11\ in^2 \\ \text{if } x=4: & 0.20\ in^2 \\ \text{if } x=5: & 0.31\ in^2 \\ \text{if } x=6: & 0.44\ in^2 \\ \text{if } x=7: & 0.60\ in^2 \\ \text{if } x=8: & 0.79\ in^2 \\ \text{if } x=9: & 1.00\ in^2 \\ \text{if } x=10: & 1.27\ in^2 \\ \text{if } x=11: & 1.56\ in^2 \\ \text{if } x=14: & 2.25\ in^2 \\ \text{if } x=18: & 4.00\ in^2 \end{cases}$$

$$d_e := h - cvr - d_{bar}(Ties_{bar}) - \frac{d_{bar}(A_{s.bar})}{2}$$

$d_e = 14\ in$ — effective depth

Flexural & Shear Demands:
These are the maximum factored moment (M_u) and shear (V_u) that the beam needs to carry found through a STAAD analysis:

$M_u := 1.9694\ kip \cdot ft$ — Flexural demand 1.25DL+1.75(LL+IM) *see SAFE Output Figure 17*

$V_u := 1.542\ kip$ — Shear demand 1.25DL+1.75(LL+IM) *see SAFE Output Figure 14*

Flexural Design (Singly Reinforced Option):
The reinforcement chosen needs to provide a nominal flexural strength that exceeds the maximum factored moment (M_u) found through a STAAD analysis:

$\phi_b := 0.90$ — strength reduction factor for tension controlled sections — ACI 318-11 9.3.2

$$R_{n.req'd} := \frac{M_u}{\phi_b \cdot b \cdot d_e^{\,2}} = 19\ psi$$
required coefficient for resistance for strength design

$$m := \frac{f_{y.bar}}{0.85 \cdot f'_c} = 15.69$$
stress ratio

Reference

$$\rho_{req'd} := \frac{1}{m}\left(1-\sqrt{1-\frac{2\cdot m\cdot R_{n.req'd}}{f_{y.bar}}}\right) = 0.0003$$ required ratio for strength design

$$\rho_{min} := \max\left(\frac{3\cdot\sqrt{\frac{f'_c}{psi}}}{f_{y.bar}}\cdot psi, \frac{200}{f_{y.bar}}\cdot psi\right) = 0.0034$$ required minimum ratio for flexural design — ACI 318-11 10.5.1

$A_{s.req'd} := \max\left(\rho_{req'd}, \rho_{min}\right)\cdot b\cdot d_e$ required flexural reinforcement in tension

$\max\left(\rho_{req'd}, \rho_{min}\right) = 0.0034$

$A_{s.req'd} = 0.329\ in^2$

$\#ofA_{s.bar} := 1$ number of bars to use for tensile reinforcement

$A_{se}\left(A_{s.bar}\right) = 0.44\ in^2$

$A_s := \#ofA_{s.bar}\cdot A_{se}\left(A_{s.bar}\right) = 0.440\ in^2$ provided steel reinforcement area

$$\beta_1 := \begin{cases} \text{if } f'_c \le 4000\ psi \\ \quad 0.85 \\ \text{if } f'_c > 4000\ psi \\ \quad \max\left(0.85 - 0.05\cdot\left(\frac{f'_c - 4000\ psi}{1000\ psi}\right), 0.65\right)\end{cases} = 0.83$$

ratio of the depth of rectangular stress distribution to the depth of the neutral axis — ACI 318-11 10.2.7.3

$$\rho_{prov} := \frac{A_s}{b\cdot d_e} = 0.0045$$

$$\varepsilon_y := \frac{f_{y.bar}}{E_s} = 0.0021$$ tension yielding strain limit for reinforcement steel

$$\rho_b := \frac{0.85\cdot f'_c}{f_{y.bar}}\cdot\beta_1\cdot\left(\frac{0.003\cdot E_s}{0.003\cdot E_s + f_{y.bar}}\right) = 0.0311$$ balanced strain condition

$$\rho_{max} := \frac{0.003+\varepsilon_y}{0.007}\cdot\rho_b = 0.0225$$ maximum reinforcement ratio at ε_t=0.004 which is the lowest permitted steel strain at ult strength of flexural members

ρ_{max}

$check.\rho_{max} = \text{"OK"}$

$$check.\rho_{max} := \begin{cases}\text{if } \rho_{max} > \rho_{prov} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"NOT OK"}\end{cases}$$

Check the design as follows:

$T_b := A_s\cdot f_{y.bar} = 26.4\ kip$ Tensile capacity of reinforcement steel

$$a_b := \frac{T_b}{0.85\cdot f'_c\cdot b} = 0.99\ in$$ Whitney Equivalent rectangular block depth

$$M_n := T_b\cdot\left(d_e - \frac{a_b}{2}\right) = 29.72\ kip\cdot ft$$ nominal flexural strength

$$x_b := \frac{a_b}{\beta_1} = 1.2\ in$$ the neutral axis is located x_b from the point of maximum compression

Reference

$\varepsilon_t := \left(\dfrac{d_e - x_b}{x_b}\right) \cdot 0.003 = 0.0321$ net strain in the tension steel

ACI 318-11
9.3.2
Fig. R9.3.2

$check.\varepsilon_s :=$ if $\varepsilon_t \le 0.002$ → "Compression Controlled"; if $0.002 < \varepsilon_t < 0.005$ → "Transion Range"; if $\varepsilon_t \ge 0.005$ → "Tension Controlled"

$\phi_{b.adj} :=$ if $\varepsilon_t \le 0.002$ → 0.65; if $0.002 < \varepsilon_t < 0.005$ → $0.65 + (\varepsilon_t - 0.002) \cdot \left(\dfrac{250}{3}\right)$; if $\varepsilon_t \ge 0.005$ → 0.90

$check.\varepsilon_s =$ "Tension Controlled"

$\phi_{b.adj} = 0.90$

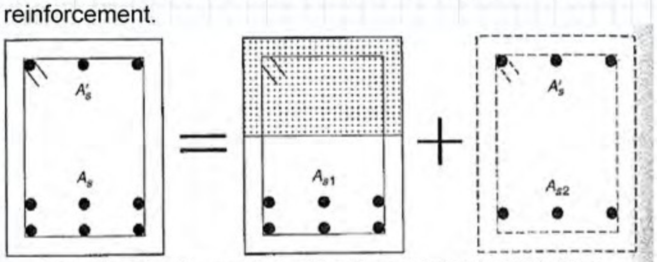
$\phi M_n := \phi_{b.adj} \cdot M_n = 26.74\ \textbf{kip} \cdot \textbf{ft}$

$\phi M_n = 26.74\ \textbf{kip} \cdot \textbf{ft}$

$check.\phi M_n :=$ if $\phi M_n > M_u$ → "OK"; else → "NOT OK" $\qquad M_u = 1.97\ \textbf{kip} \cdot \textbf{ft}$

$check.\phi M_n =$ "OK"

Flexural Design (Doubly Reinforced Beam Option):
The reinforcement chosen needs to provide a nominal flexural strength that exceeds the maximum factored moment (M_u) found through a STAAD analysis. The capacity of a doubly reinforced beam is the superposition of (1) a singly reinforced beam with an area of steel A_{s1}, and (2) an tension-compression steel section, with A'_s as compression reinforcement and A_{s2} as tensile reinforcement.



(a) Doubly-reinforced beam = singly-reinforced beam + tension-compression steel

Choose a compresion side reinforcement size to check:

$A_{s.bar} = 6$ Tension side steel reinforcement bar size (*matches single side beam reinforcement design*)

$A'_{s.bar} := 4$ Compresion side steel reinforcement bar size

$d_{bar}(A'_{s.bar}) = 0.5\ \textbf{in}$ $\qquad d_{bar}(A_{s.bar}) = 0.75\ \textbf{in}$ bar diameter

$A_{se}(A'_{s.bar}) = 0.20\ \textbf{in}^2$ $\qquad A_{se}(A_{s.bar}) = 0.44\ \textbf{in}^2$ bar area

$\#ofA'_{s.bar} := 1$ number of bars to use for compression reinforcement

$\#ofA_{s.bar} = 1$ number of bars to used as tensile reinforcement

Reference

$A'_s := \#ofA'_{s.bar} \cdot A_{se}(A'_{s.bar}) = 0.200\ in^2$ — provided compression steel reinforcement area

$A_s = 0.44\ in^2$ — total area of tension steel

ACI 318-11
7.11.1
7.10.5.1
7.10.5.2

With the use of compression reinforcement there is the need for lateral bracing provided by closed ties/stirrups. Per ACI 318 7.11.1 and 7.10.5, the following ties and spacing will need to be provided for the size of compression bars selected:

$$A_{s.ties} := \text{if } A'_{s.bar} \le 10 \;\; \| \; 3 \;\; \text{else} \;\; \| \; 4$$

$A_{s.ties} = 3$

$d_{bar}(A_{s.ties}) = 0.375\ in$

$A_{se}(A_{s.ties}) = 0.11\ in^2$

The required spacing of these ties needs to be the smallest of the following quantities set by ACI 318-11 7.10.5.2

$$min(16 \cdot d_{bar}(A'_{s.bar}), 48 \cdot d_{bar}(A_{s.ties}), b) = 7\ in$$

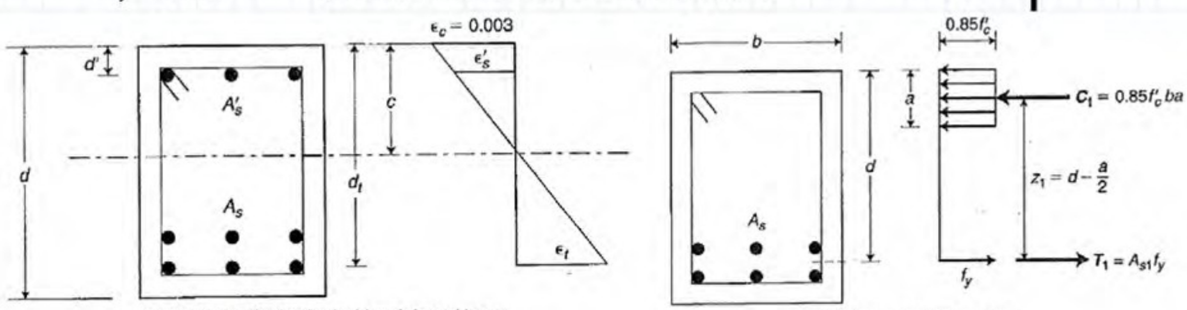
$$S_{ties} := \text{Floor}(min(16 \cdot d_{bar}(A'_{s.bar}), 48 \cdot d_{bar}(A_{s.ties}), b), 1\ in) = 7\ in$$

$$d' := cvr + d_{bar}(A_{s.ties}) + \frac{d_{bar}(A'_{s.bar})}{2} = 2.63\ in$$

distance from center of compression steel to the compression edge of the beam

Step 1: Let us assumed that the compression steel has yielded ($\varepsilon'_s \ge \varepsilon_y$) before the concrete in compression has reached its ultimate strain. Therefore ($f'_s = f_{y.bar}$)

$\varepsilon_y = 0.0021$ — yielding strain of reinforcement steel (fy/E)



(b) Strain distribution in doubly-reinforced beam

(c) Concrete-steel couple

$A_{s2} := A'_s = 0.2\ in^2$ — area of tension steel required to work with the area of compression steel

$A_{s1} := A_s - A'_s = 0.24\ in^2$ — area of tension steel for concrete-steel force couple

$\frac{A_{s1}}{b \cdot d_e}$

Step 2: Calculate the depth of the compression zone and location of neutral axis:

$$a := \frac{A_{s1} \cdot f_{y.bar}}{0.85 \cdot f'_c \cdot b} = 0.54\ in \qquad c := \frac{a}{\beta_1} = 0.65\ in$$

Reference

Step 3: Determine the strain levels of the tensile steel (ε_t) and the compression steel (ε'_s) from the similarity of triangles (see figure above):

$$d_t := h - cvr - d_{bar}\left(A_{s.ties}\right) - \frac{d_{bar}\left(A_{s.bar}\right)}{2} = 14.25\ in$$

effective depth of the extreme tension steel

$$\varepsilon_t := \frac{0.003 \cdot \left(d_t - c\right)}{c} = 0.0626$$

$$\varepsilon_y = 0.0021$$

$$check.\varepsilon_t := \begin{array}{l} \text{if } \varepsilon_t \le \varepsilon_y \\ \quad \text{"Compression Controlled"} \\ \text{if } \varepsilon_y < \varepsilon_t < 0.005 \\ \quad \text{"Transion Range"} \\ \text{if } \varepsilon_t \ge 0.005 \\ \quad \text{"Tension Controlled"} \end{array}$$

$$\phi_{b.adj} := \begin{array}{l} \text{if } \varepsilon_t \le \varepsilon_y \\ \quad 0.65 \\ \text{if } \varepsilon_y < \varepsilon_t < 0.005 \\ \quad 0.65 + \left(\varepsilon_t - 0.002\right) \cdot \left(\frac{250}{3}\right) \\ \text{if } \varepsilon_t \ge 0.005 \\ \quad 0.90 \end{array}$$

$check.\varepsilon_t =$ "Tension Controlled"

$\phi_{b.adj} = 0.90$

Step 4: Determine the strain levels of the compression steel (ε'_s) from the similarity of triangles (see figure above) to see if the steel yields when the strain in the concrete reaches 0.003:

$$\varepsilon'_s := \frac{0.003 \cdot \left(c - d'\right)}{c} = -0.0091 \qquad \varepsilon_y = 0.0021$$

$$check.\varepsilon'_s := \begin{array}{l} \text{if } \varepsilon'_s \ge \varepsilon_y \\ \quad \text{"Case 1"} \\ \text{if } \varepsilon'_s < \varepsilon_y \\ \quad \text{"Case 2"} \end{array}$$

Case 1: Compression reinforcement yields
Case 2: Compression reinforcement does not yield

$check.\varepsilon'_s =$ "Case 2"

Reference

Step 5:

CASE 2

Because ($\varepsilon'_s < \varepsilon_y$), the compression steel did not yield when the strain at the extreme compression edge on the concrete edge reached 0.003. The stress in the compression steel (f'_s) can be calculated as:

$f'_s := \varepsilon'_s \cdot E_s = -263324$ ***psi*** stress in the compression steel

The assumption made in step 1 was not correct. The force provided by the compression steel is less than was assumed. Hence a smaller portion of tensile steel will work in the compression steel - tension steel couple, and a new location has been determined for the Neutral Axis. Find it solving the following equation based on the equilibrium of the total section tension and compression forces:

Guess Values

$c_{c2} := \dfrac{h}{2}$ $h = 17$ ***in*** this is the guess value for the solver

Constraints

$$\left(0.85 \cdot f'_c \cdot b \cdot \beta_1\right) \cdot c_{c2}^{\ 2} + \left(0.003 \cdot E_s \cdot A'_s - A_s \cdot f_{y.bar}\right) \cdot c_{c2} - 0.003 \cdot d' \cdot E_s \cdot A'_s = 0$$

Solver

find $\left(c_{c2}\right) = 1.656$ ***in***

$c_{c2} := 1.656$ ***in*** actual location of neutral axis (copy value from solver)

Step 6 & 6a: Once the updated distance to the neutral axis is known, determine the net tensile strain in the extreme layer of steel ($\varepsilon_{t.c2}$)

$$\varepsilon_{t.c2} := \frac{0.003 \cdot \left(d_t - c_{c2}\right)}{c_{c2}} = 0.0228 \qquad \varepsilon_y = 0.0021$$

$$check.\varepsilon_{t.c2} := \begin{cases} \text{if } \varepsilon_{t.c2} \le \varepsilon_y & \text{“Compression Controlled”} \\ \text{if } \varepsilon_y < \varepsilon_{t.c2} < 0.005 & \text{“Transion Range”} \\ \text{if } \varepsilon_{t.c2} \ge 0.005 & \text{“Tension Controlled”} \end{cases}$$

$$\phi_{b.case2} := \begin{cases} \text{if } \varepsilon_{t.c2} \le \varepsilon_y & 0.65 \\ \text{if } \varepsilon_y < \varepsilon_{t.c2} < 0.005 & 0.65 + \left(\varepsilon_{t.c2} - 0.002\right) \cdot \left(\dfrac{250}{3}\right) \\ \text{if } \varepsilon_{t.c2} \ge 0.005 & 0.90 \end{cases}$$

$check.\varepsilon_{t.c2} =$ “Tension Controlled”

$\phi_{b.case2} = 0.90$

Step 7: Calculate the stress in the compression steel ($f'_{s.case2}$):

$$f'_{s.case2} := \frac{0.003 \cdot \left(c_{c2} - d'\right)}{c_{c2}} \cdot E_s = -50.91 \ \textbf{\textit{ksi}}$$

then calculate the depth of the equivalent stress block (a_{case2})

$a_{case2} := \beta_1 \cdot c_{c2} = 1.37$ ***in***

Reference

Step 8: Calculate $M_{n1.case2}$ and $M_{n2.case2}$, the nominal resisting moments of the *concrete-tensile steel couple* and the *compression steel-tensile steel* couple, respectively:

$$M_{n1.case2} := \left(0.85 \cdot f'_c \cdot b \cdot a_{case2}\right) \cdot \left(d_t - \frac{a_{case2}}{2}\right)$$

$M_{n1.case2} = 41.36\ \mathbf{kip \cdot ft}$ — *concrete-tensile steel couple* nominal resisting moment

$$M_{n2.case2} := A'_s \cdot f'_{s.case2} \cdot \left(d_t - d'\right)$$

$M_{n2.case2} = -9.86\ \mathbf{kip \cdot ft}$ — *compression steel-tensile steel couple* nominal resisting moment

The total nominal resisting moment is:

$$\phi M_{n.case2} := \phi_{b.case2} \cdot \left(M_{n1.case2} + M_{n2.case2}\right) = 28.34\ \mathbf{kip \cdot ft}$$

$M_u = 1.97\ \mathbf{kip \cdot ft}$

$\phi M_{n.case2} = 28.34\ \mathbf{kip \cdot ft}$

check.$\phi M_{n.case2}$:= if $\phi M_{n.case2} > M_u$
  "OK"
else
  "NOT OK"

check.$\phi M_{n.case2}$ = "OK"

Shear Capacity Check:
The concrete section nominal shear strength will be checked if it exceeds exceeds the maximum factored shear (V_u) found through the SAFE analysis:

$\phi_v := 0.75$ — ACI 318-11 9.3.2.3

$\lambda_{conc} := 1.0$ — ACI 318-11 9.6.1

$$V_c := 2 \cdot \lambda_{conc} \cdot \sqrt{\frac{f'_c}{\mathbf{psi}}} \cdot \frac{b}{\mathbf{in}} \cdot \frac{d_t}{\mathbf{in}} \cdot \mathbf{lbf} = 13.38\ \mathbf{kip}$$

for members subject to shear and flexure only — ACI 318-11 11.2.1.1 (11-3)

$\phi V_c := \phi_v \cdot V_c = 10.04\ \mathbf{kip}$

check.ϕV_c := if $\phi V_c > V_u$
  "OK"
else
  "NOT OK"

$V_u = 1.54\ \mathbf{kip}$

check.ϕV_c = "OK"

$$V_s := \frac{2 \cdot A_{se}\left(A_{s.ties}\right) \cdot f_{y.bar} \cdot d_t}{S_{ties}} = 26.87\ \mathbf{kip}$$

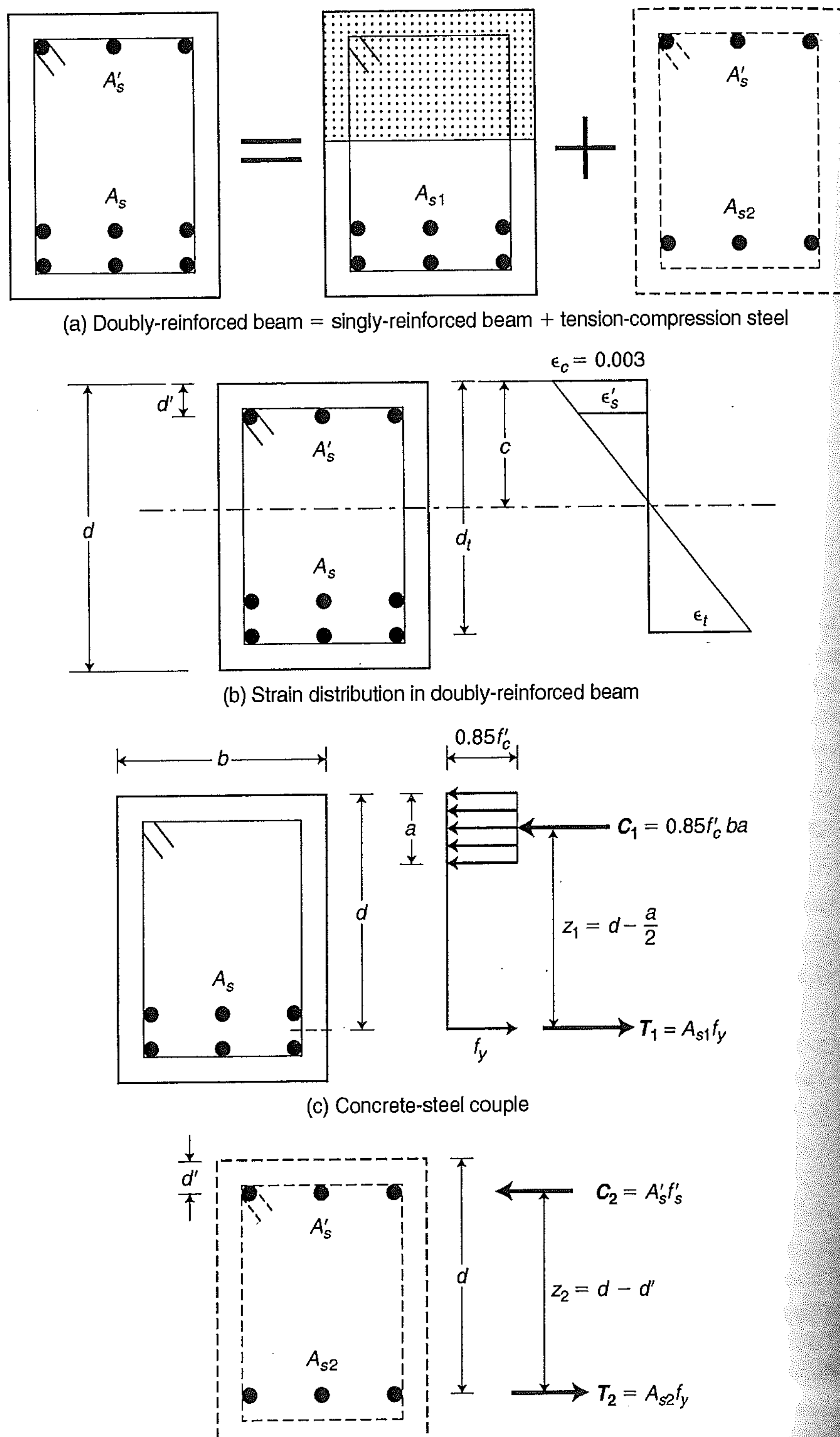
Shear capacity of ties used as shear reinforcement — ACI 318-11 11.4.7.2 (11-15)

$\phi V_n := \phi_v \cdot \left(V_c + V_s\right) = 30.19\ \mathbf{kip}$

nominal combined shear capacity — 11.1.1 (11-2)

check.ϕV_n := if $\phi V_n > V_u$
  "OK, but shear reinf req'd"
else
  "NOT OK"

check.ϕV_n = "OK, but shear reinf req'd"



(a) Doubly-reinforced beam = singly-reinforced beam + tension-compression steel

(b) Strain distribution in doubly-reinforced beam

(c) Concrete-steel couple

(d) Steel-steel couple

FIGURE 3-16 Analysis of doubly-reinforced beams.

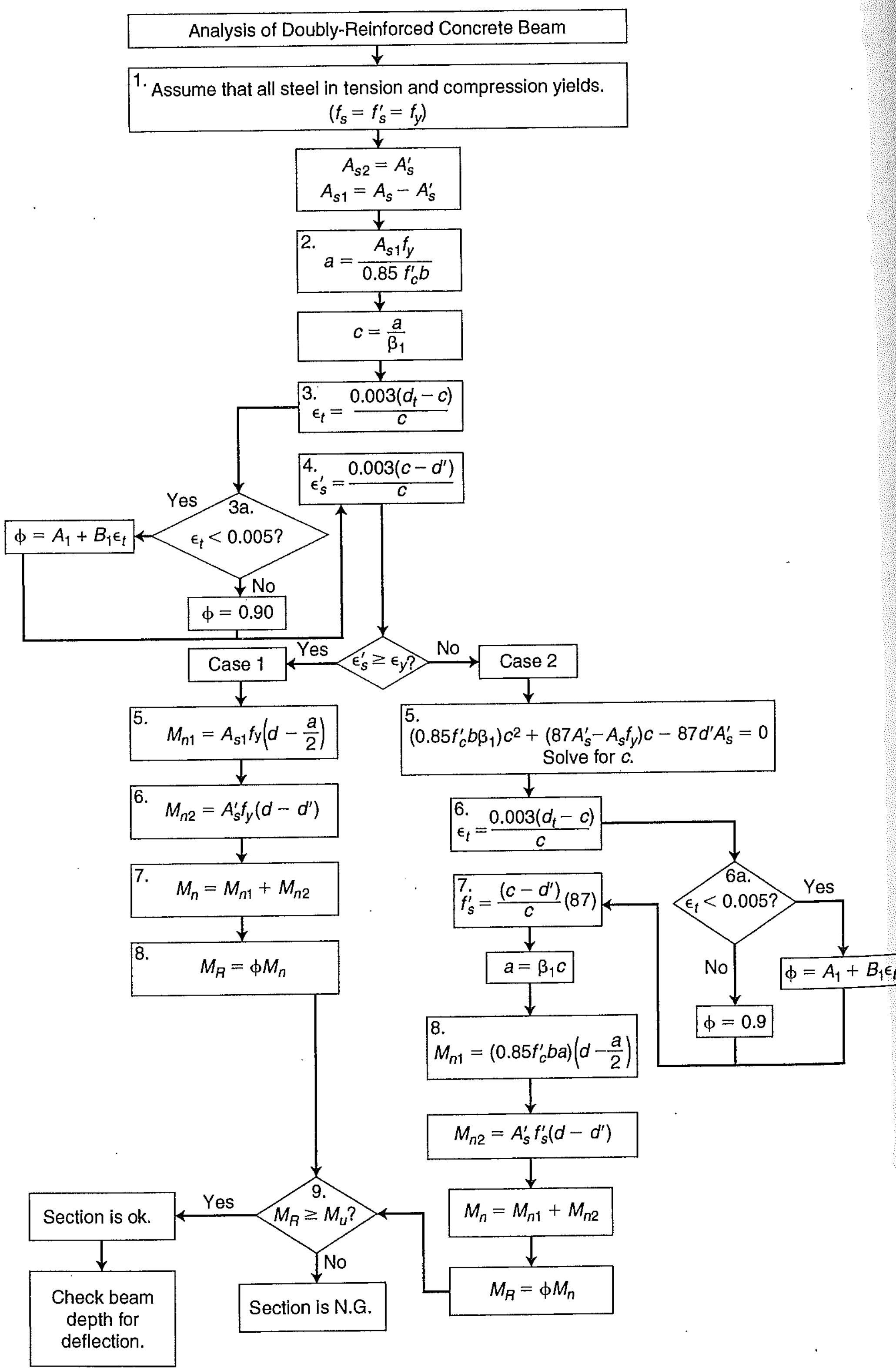


FIGURE 3–17 Analysis of doubly-reinforced beams.

SECTION 300
STEM WALL

MWH now part of Stantec

By I. BATILOV Date 2/1/18 Client UNLV Sheet 5 of

Chkd. By D. ROUNDS Description DRT RUMBLE STRIP Job No. 181307096

Design Task STEM WALL DESIGN FOR SHEAR/FLEXURE DEMAND DUE TO BRAKING FORCE

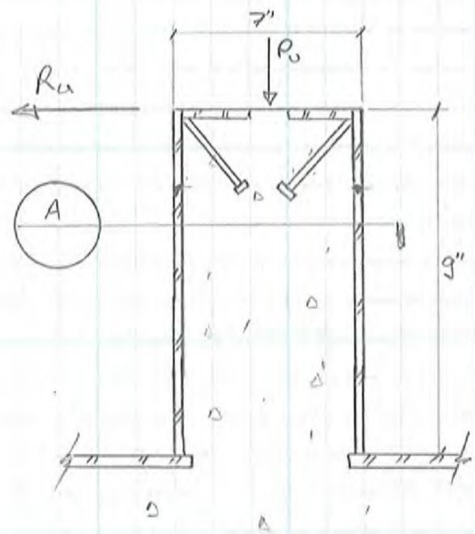
DESIGN VERTICAL LEG OF CONCRETE BOX FOR SHEAR AND FLEXURE INDUCED BY BREAKING FORCE (BR) CALCULATED ON PAGE 2.

F_{BR} = 4.0 k APPLIED OVER THE WIDTH OF TIRE CONTACT AREA OF 20"

REFERENCES

AASHTO LRFD 3.6.1.2-5

THE VERTICAL LEG IS FRAMED w/ 1/8" PL SO THAT WILL BE CONSIDERED AS REINFORCEMENT



FOR 1.25 DC + 1.75 (LL + IM + BR)

AXIAL DIRECTION (FOR 20" STRIP)

$$DL = \left(\frac{7''}{12''/_{ft}}\right)\left(\frac{20''}{12''/_{ft}}\right)\left(\frac{9''}{12''/_{ft}}\right)(150\,pcf)$$

$= 109.4$ LBS $= 0.109^k$

$LL + IM = 1.33(16^k) = 21.28^k$

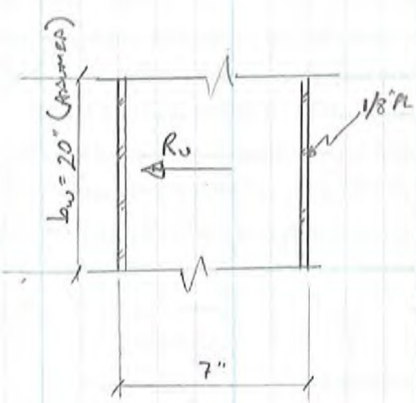
$P_u = 1.25(0.109^k) + 1.75(21.28^k)$
$= 37.4^k$

LATERAL DIRECTION

$BR = 4^k$

$R_u = 1.75(4.0^k) = 7.0^k$

$M_u = R_u(9'') = (7.0^k)(9'') = 63$ k·in



CHECK CONCRETE SHEAR CAPACITY

PER ACI 318-11, 11.2.1.1, FOR SECTIONS w/ SIGNIFICANT AXIAL COMPRESSION IN ADDITION TO FLEXURE & SHEAR

ACI 318-11 11.2.1.1 Eq. (11-4)

$$V_c = 2\left(1 + \frac{N_u}{200 A_g}\right)\sqrt{f'_c}\, b_w d$$

N_u = AXIAL COMPRESSION = $P_u = 37.4^k = 37400$ LBS

b_w = 20" (EQUIVALENT TO TIRE CONTACT WIDTH)

d = ASSUME 1/2 of SECTION THICKNESS = 7"/2 = 3.5"

ϕ = 0.75 (ACI 318 9.3.2.3) 9.3.2.3

f'_c = 4500 psi (CONCRETE COMPRESSIVE STRENGTH)

A_g = GROSS-CROSS-SECTIONAL AREA = $b_w(7'') = (20'')(7'') = 140\ in^2$

$$\phi V_c = (0.75)\,2\left(1 + \frac{37400\,LBS}{200(140\,in^2)}\right)\sqrt{4500\,psi}\,(20'')(3.5'') = 16452\ \#$$
$$= 16.45^k > R_u = 7^k \quad OK!$$

5 squares per inch

REFERENCES

THE LATERAL LOAD WILL BE EQUATED TO AN ECCENTRIC AXIAL LOAD INDUCING THE SAME FLEXURAL DEMAND, M_u.

$$e = \frac{M_u}{P_u} = \frac{63^{k\cdot in}}{37.4^k} = 1.684\,in$$

THE LOAD CAPACITY OF A TIED COLUMN UNDER A CONCENTRIC LOAD IS:

$$\phi P_n = 0.8\phi\left[0.85 f'_c\left(A_g - A_{st}\right) + f_y A_{st}\right]$$

$$\phi P_n = 0.8(0.65)\left[0.85(4.5ksi)(140in^2 - 5in^2) + (36ksi)(5in^2)\right]$$

$$= 0.8(0.65)\left[516.38^k + 180^k\right]$$

$$= 362.1^k > P_u \quad \text{OK!}$$

A_g = GROSS AREA OF COL
= (20")(7")
= 140 in²

A_{st} = Area of STEEL
(ASSUME ADJACENT 1/8" PL)
= (1/8")(20")(2 PL)
= 5 in²

f_y = 36 ksi (A36 STEEL)
60 ksi (GR.60 REINF)

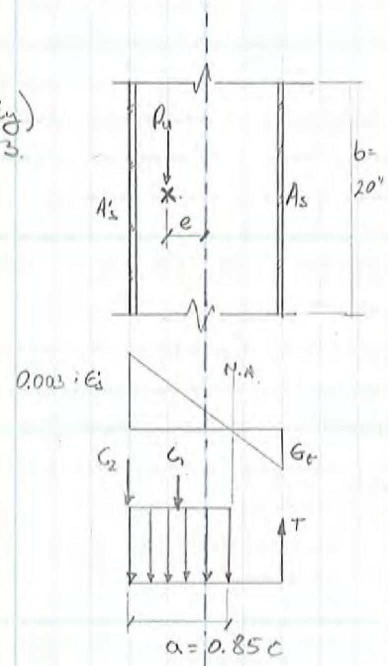
ϕ = 0.65 (TIED COLUMNS)

f'_c = 4500 psi

THE NOMINAL AXIAL LOAD STRENGTH, P_n, AND NOMINAL FLEXURAL STRENGTH, M_n, FOR THE SHORT COLUMN WILL BE FOUND, CONSIDERING THE P_u LOAD IS APPLIED AT AN $e = 1.684$".

THE STRESS AND STRAIN IN THE STEEL ARE PROPORTIONAL TO THE YIELD POINT. LET'S ASSUME TENSILE STEEL (A_s) HAS NOT REACHED YIELD ($\varepsilon_t < \varepsilon_y$) WHEN CONCRETE REACHES COMPRESSIVE STRAIN OF 0.003. YIELD STRAIN (ε_y) FOR GR. 36 PL IS f_y/E_s = 36 ksi/29,000 ksi = 0.00124 AND BECAUSE COMPRESSION STEEL (A'_s) IS close TO COMPRESSION EDGE, WE CAN ASSUME STRAIN IN COMPRESSION STEEL IS MORE THAN YIELD ($\varepsilon'_s > \varepsilon_y$)

THEREFORE $\varepsilon'_s > \varepsilon_y \rightarrow f'_s = f_y$
$\varepsilon_t < \varepsilon_y \rightarrow f_s < f_y$



THE STRAINS IN THE TENSION & COMPRESSION STEEL DEPEND ON THEIR LOCATION TO THE NEUTRAL AXIS, (c). FROM SIMILARITY OF TRIANGLES:

$$\frac{\varepsilon_t}{0.003} = \frac{d-c}{c} \rightarrow \varepsilon_t = 0.003\left(\frac{d-c}{c}\right)$$

THE STRESS IN THE TENSILE STEEL IS:

$$f_s = E_s\varepsilon_t = (29000ksi)(0.003)\left(\frac{d-c}{c}\right) = 87\left(\frac{d-c}{c}\right)$$

$$d = 7" - 0.5\left(\frac{1"}{8}\right) = 6.9375"$$

REFERENCES

THE COMPRESSION AND TENSILE FORCES ACTING ON THE SECTION ARE:

COMPRESSION FORCES:

$$C_1 = 0.85 f'_c a b = 0.85\,(4.5_{ksi})(0.85c)(20") = 65.03\,c$$

$$C_2 = A'_s\,(f_y - 0.85 f'_c) = (1/8")(20")(36_{ksi} - 0.85(4.5)) = 80.44^k$$

TENSILE FORCE, T:

$$T = f_s A_s \text{, SUBSTITUTING } f_s \text{ w/ } 87\left(\frac{d-c}{c}\right)$$

$$T = 87\left(\frac{6.9375" - c}{c}\right)(2.5\ in^2)$$

$$T = 217.5\left(\frac{6.9375" - c}{c}\right) \text{ kips}$$

EQUILIBRIUM REQUIRES THAT THE SUM OF FORCES BE EQUAL TO ZERO

$$P_n - C_1 - C_2 + T = 0 \rightarrow P_n = C_1 + C_2 - T$$

$$P_n = 65.03c + 80.44^k - 217.5\left(\frac{6.9375" - c}{c}\right)^{kips}$$

IN ADDITION, SUM OF MOMENTS NEED TO EQUAL ZERO, TAKING THE MOMENTS ABOUT THE LOCATION OF TENSILE STEEL (A_s) FOR SIMPLICITY:

$$P_n(3.4375" + \overset{e}{1.684"}) - C_1\left(d - \frac{a}{2}\right) - C_2(6.875") = 0$$

$$P_n(5.1215") = C_1\left(6.9375" - \frac{0.85c}{2}\right) + C_2(6.875")$$

$$P_n = \left(\frac{1}{5.1215"}\right)\left[65.03c\left(6.9375" - \frac{0.85c}{2}\right) + 80.44^k(6.875")\right]$$

$$P_n = \left(\frac{1}{5.1215"}\right)\left[451.146c - 27.638c^2 + 553.025_{k\text{-}in}\right]$$

$$P_n = -5.3965c^2 + 88.09c + 107.98^k$$

EQUATING THE TWO EXPRESSIONS OF P_n:

$$65.03c + 80.44^k - 217.5\left(\frac{6.9375" - c}{c}\right) = -5.3965c^2 + 88.09c + 107.98^k$$

$$\frac{-1508.91}{c} + 217.5 = -5.3965c^2 + 23.06c + 27.54^k$$

$$-1508.91 = -5.3965c^3 + 23.06c^2 - 189.96c$$

$$0 = 5.3965c^3 - 23.06c^2 + 189.9c - 1508.91$$

SOLVING FOR c, USING MATHCAD SOLVER,

$$c = 6.07\ in$$

REFERENCES

SUBSTITUTING C IN ONE OF THE P_n EQUATIONS:

$$P_n = -5.3965(6.07)^2 + 88.09(6.07'') + 107.98^k$$

$$= 443.85^k$$

KNOWING C AND P_n, MUST CHECK ASSUMPTIONS. CALCULATING STRAIN IN COMPRESSION STEEL (ϵ'_s) VIA SIMILARITY OF TRIANGLES

$$\frac{\epsilon'_s}{0.003} = \frac{c - 0.5(1/8'')}{c} \rightarrow \epsilon'_s = 0.003\left(\frac{6.07'' - 0.0625''}{6.07''}\right)$$

$$\epsilon'_s = 0.00297 > \epsilon_y \left(f_y/E_s\right) = 0.00124$$

STRAIN IN COMPRESSION STEEL IS MORE THAN THE YIELD STRAIN; THEREFORE STRESS IS EQUAL TO THE YIELD STRESS AS ASSUMED, $f'_s = f_y$ AND COMPRESSION STEEL HAS YIELDED.

NOW TO FIND LEVEL OF STRAIN IN THE TENSILE STEEL:

$$\epsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{6.9375'' - 6.07''}{6.07''}\right)$$

$$\epsilon_t = 0.00043 < \epsilon_y = 0.00124$$

HENCE SECOND ASSUMPTION THAT TENSILE STEEL (A_s) HAS NOT YIELDED WAS CORRECT. THE NOMINAL MOMENT CAPACITY @ THIS ECCENTRICITY OF LOAD IS:

$$M_n = P_n e = (443.85^k)(1.684'')$$
$$= 747.44 \text{ k-in}$$
$$= 62.29 \text{ k-ft}$$

AS NOTED EARLIER, FOR COMPRESSION CONTROLLED SECTIONS W/ NO SPIRAL REINFORCEMENT, $\phi = 0.65$ — ACI 318-11 9.3.2.2

$$\phi P_n = (0.65)(443.85^k) = \underline{\underline{288.5^k > P_u = 37.4^k}} \quad \text{OK!}$$

$$\phi M_n = (0.65)(747.44 \text{ k-in}) = \underline{\underline{485.8 \text{ k-in} > M_u = 63 \text{ k-in}}} \quad \text{OK!}$$

CHECK THAT FILLET WELD FOR 1/8" PL CAN TRANSFER TENSILE FORCE DEMAND:

$$T = 217.5\left(\frac{6.9375" - 6.07"}{6.07"}\right) = 31.1^{K} \quad \text{DEMAND ON WELD}$$

TRY 1/8" FILLET WELD: AISC 360-10 SECTION 8 (8-2a)

$$\phi R_n = 1.392\, D\, l \;,\quad D = 2 \text{ (FOR (2) SIXTEENTHS)}$$
$$l = 20" \text{ (TIRE CONTACT AREA WIDTH)}$$
$$= 1.392\,(2)(20")$$
$$= 55.68\,K$$

$$1/8" \text{ FILLET } \phi R_n = 55.68^{K} > T = 31.1^{K} \quad \text{ok!}$$

CHECK 6" STEM WALL FOR SHEAR AT ENDS

LONGITUDINAL BOX END STEM WALLS ARE 6" WIDE. THEY ARE CHECKED FOR SHEAR PER ACI 318-11, 11.2.1.1 — ACI 318-11 11.2.1.1 Eq. (11-4)

$$V_c = 2\left(1 + \frac{N_u}{200 A_g}\right)\sqrt{f'_c}\; b_w d$$

N_u = AXIAL COMPRESSION, P_u = 37400 LBS
b_w = 20" (TIRE CONTACT AREA WIDTH)
d = 1/2 OF SECTION WHERE CAR IS CENTERED
$6"/2 = 3"$
$\phi = 0.75$ (ACI 318 9.3.2.3)
f'_c = 4500 psi
A_g = GROSS CROSS-SECTIONAL AREA, $(6")(20") = 120 in^2$

$$\phi V_c = (0.75)\,2\left(1 + \frac{37400\,LBS}{200(120 in^2)}\right)\sqrt{4500 psi}\,(20")(3")$$

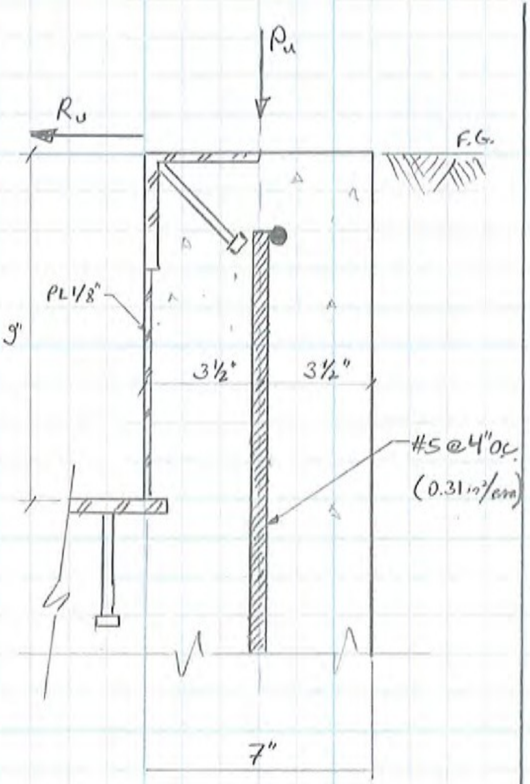
$$\phi V_c = 15{,}446\# = 15.4^{K} > R_v = 7^{K} \quad \text{OK!}$$

5 squares per inch

REFERENCES

THE 6" WIDE STEM WALLS ON THE OUTER EDGES OF THE BOX NEED TO BE CHECKED FOR AXIAL / FLEXURE DEMANDS OF THE TRAFFIC LIVE LOAD CONSIDERING BRAKING FORCE.

6" STEM WALL IS FRAMED W/ 1/8" PL AND #5 BAR CENTERED IN SECTION. (SEE FIGURE)



AXIAL LOAD, P_u (FOR 20" TIRE AREA)

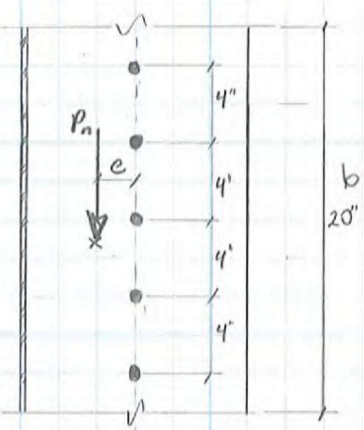
$$DL = \left(\frac{7"}{12"/ft}\right)\left(\frac{20"}{12"/ft}\right)\left(\frac{9"}{12"/ft}\right)\underbrace{(150\ pcf)}_{}^{\gamma_{conc}}$$

$$= 109.4^{\#} = 0.109^{k}$$

$$LL + IM = 1.33(16^k) = 21.28^k$$

FOR $1.25\,DL + 1.75\,(LL + IM + BR)$

$$P_u = 1.25(0.109^k) + 1.75(21.28^k) = 37.4^k$$



LATERAL DIRECTION, R_u (FOR 20" TIRE AREA)

BRAKING FORCE, BR = 4k (SEE SECTION 100)

FOR $1.25\,DL + 1.75\,(LL + IM + BR)$

$$R_u = 1.75(4^k) = 7^k$$

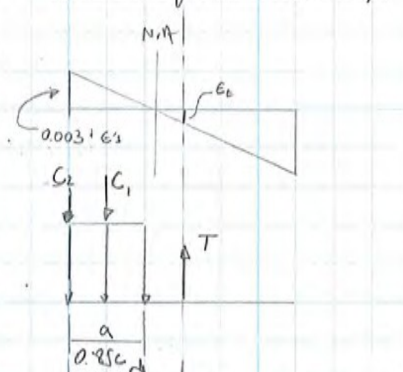
$$M_u = R_u(9") = (7^k)(9") = 63^{k\cdot in}$$

LATERAL LOAD EQUATED TO AN ECCENTRIC AXIAL LOAD INDUCING THE SAME FLEXURAL DEMAND, M_u

$$e = \frac{M_u}{P_u} = \frac{63^{k\cdot in}}{37.4^k} = 1.684\ in$$

TENSILE STEEL, A_s

$$A_s = \left(\frac{20"}{4"/bar}\right)(0.31\ in^2) = 1.55\ in^2$$



$d = 7"/2 = 3.5"$ SINCE BAR IS CENTERED

REFERENCES

IDEALIZING A 20" SEGMENT OF THE 6" STEM WALL AS A TIED COLUMN UNDER A CONCENTRIC LOAD:

$$\phi P_n = 0.8\phi\left[0.85 f'_c (A_g - A_{ST}) + f_y A_{ST}\right], \quad A_g = (20")(7") = 140\ in^2$$

A_{ST} = AREA OF STEEL, 1/8" PL + #5 @ 4" O.C FOR 20"

$= (1/8")(20") + \left(\frac{20"}{4"/BAR}\right)(0.31\ in^2)$

$= 4.05\ in^2$

$$= 0.8(0.65)\left[0.85(4.5ksi)(140 in^2 - 4.05 in^2) + (36ksi)(4.05 in^2)\right]$$

$$= 0.8(0.65)\left[520.01^K + 145.8^K\right]$$

$$\phi P_n = 346.2^K > P_u = 37.4^K \ \text{OK!}$$

$f'_c = 4500\ psi = 4.5\ ksi$

$f_y = 36\ ksi$ (A36 STEEL)

$\phi = 0.65$ (OTHER REINF. MEMBERS) ACI 318-11, 9.3.2.2

THE NOMINAL AXIAL LOAD STRENGTH, P_n, AND NOMINAL FLEXURAL STRENGTH, M_n, FOR A SHORT COLUMN CONSIDERING THE P_u LOAD IS APPLIED AT AN ECCENTRICITY, e OF 1.684 in

THE STRESS AND STRAIN IN THE STEEL ARE PROPORTIONAL TO THE YIELD POINT. ASSUMING TENSILE STEEL (A_s) HAS NOT REACHED YIELD STRAIN ($\epsilon_t < \epsilon_y$) AT THE TIME CONCRETE REACHES COMPRESSIVE STRAIN OF 0.003, AND THAT YIELD STRAIN FOR GR. 60 REBAR IS: $f_y/E_s = \frac{60ksi}{29000ksi} = 0.0021$

AND BECAUSE COMPRESSION STEEL (A'_s) IS AT COMPRESSION EDGE, WE CAN ASSUME STRAIN IN COMPRESSION STEEL IS MORE THAN ϵ_y (YIELD) ($\epsilon'_s > \epsilon_y$)

THEREFORE, $\epsilon'_s > \epsilon_y \rightarrow f'_s = f_y$

$\epsilon_t < \epsilon_y \rightarrow f_s < f_y$

STRAIN IN THE TENSION / COMPRESSION STEEL DEPEND ON THEIR LOCATION TO NEUTRAL AXIS (c), SO FROM SIMILARITY OF $\triangle$'s

$$\frac{\epsilon_t}{0.003} = \frac{d-c}{c} \rightarrow \epsilon_t = 0.003\left(\frac{d-c}{c}\right)$$

STRESS IN TENSILE STEEL IS:

$$f_s = E_s \epsilon_t = (29000ksi)\,0.003\left(\frac{d-c}{c}\right) = 87\left(\frac{d-c}{c}\right)$$

COMPRESSION/TENSION FORCES ACTING ON SECTION ARE:

CONCRETE: $C_1 = 0.85 f'_c\, a\, b = 0.85(4.5ksi)(0.85c)(20") = 65.03\,c$

COMPR. STEEL: $C_2 = A'_s(f_y - 0.85 f'_c) = (1/8")(20")(36ksi - 0.85(4.5ksi))$

$= 80.44^K$

REFERENCES

TENSILE FORCE, T:

$T = f_s A_s$, SUBSTITUTING f_s w/ $87\left(\frac{d-c}{c}\right)$

$$T = 87\left(\frac{3.5''-c}{c}\right)(1.55\,in^2) = 134.85\left(\frac{3.5''-c}{c}\right)\text{ kips}$$

SUM OF THE FORCES MUST EQUAL 0,

$$P_n - C_1 - C_2 + T = 0 \rightarrow P_n = C_1 + C_2 - T$$

$$P_n = 65.03c + 80.44^k - 134.85\left(\frac{3.5''-c}{c}\right) \quad (1)$$

ADDITIONALLY, SUM OF MOMENTS NEED TO EQUAL ZERO. TAKING MOMENT ABOUT THE LOCATION OF THE TENSILE STEEL (A_s)

$$P_n\left(1.684''\right) - C_1\left(d - \frac{a}{2}\right) - C_2\left(3.5'' - 0.5(1/8'')\right) = 0$$

$$(1.684'')P_n = 65.03c\left(3.5 - \frac{0.85c}{2}\right) + (80.44^k)(3.4375'')$$

$$P_n = \left(\frac{1}{1.684''}\right)\left[227.61c - 27.64c^2 + 276.51\,k\cdot in\right]$$

$$P_n = -16.41c^2 + 135.16c + 164.2^k \quad (2)$$

EQUATING THE TWO EXPRESSIONS (1) : (2)

$$65.03c + 80.44^k - 134.85\left(\frac{3.5''-c}{c}\right) = -16.41c^2 + 135.16c + 164.2^k$$

$$\frac{-471.98}{c} + 134.85 = -16.41c^2 + 70.13c + 83.76^k$$

$$-471.98 = -16.41c^3 + 70.13c^2 - 51.09c$$

$$0 = -16.41c^3 + 70.13c^2 - 51.09c + 471.98^{k\cdot in}$$

$$0 = 16.41c^3 - 70.13c^2 + 51.09c - 471.98^{k\cdot in}$$

SOLVING FOR c, VIA MATHCAD

$$c = 4.85\text{ in}$$

SUBSTITUTING c, IN ONE OF THE P_n EQUATIONS

$$P_n = -16.41(4.85\text{ in})^2 + 135.16(4.85) + 164.2^k = 433.72^k$$

w/ c AND P_n, CHECK ASSUMPTIONS. THE STRAIN IN COMPRESSION STEEL CAN BE FOUND VIA SIMILAR TRIANGLES:

$$\frac{\epsilon_s'}{0.003} = \frac{c-d'}{c} \rightarrow \epsilon_s' = 0.003\left(\frac{4.85'' - 0.5(1/8'')}{4.85''}\right)$$

$$\epsilon_s' = 0.00296 > 0.00124$$

$$\epsilon_y \text{ FOR A36 STEEL} = f_y/E_s = 36/29000 = 0.00124$$

MWH now part of Stantec

By: I. BATILON Date: 2/23/18 Client: UNLV Sheet: 13 of
Chkd. By: D. KOUNDS Description: DRT RUMBLE STRIP Job No. 181307096
Design Task: 7" STEM WALL AT ENDS.

REFERENCES

STRAIN IN (A_s') MORE THAN YIELD STRAIN (ϵ_y); THEREFORE STRESS f_s' IS EQUAL TO YIELD STRESS AS ASSUMED, $f_s' = f_y$, SO COMPRESSION STEEL HAS YIELDED.

LEVEL OF STRAIN IN TENSION STEEL:

$$\epsilon_t = 0.003\left(\frac{d-c}{c}\right) = 0.003\left(\frac{3.5'' - 4.85''}{4.85''}\right)$$

$$= -0.00084 < \epsilon_y = 0.0021$$

SECOND ASSUMPTION THAT TENSILE STEEL HAS NOT YIELDED WAS ALSO CORRECT, $f_s < f_y$. THE NOMINAL MOMENT CAPACITY OF THE SHORT COLUMN W/ THE GIVEN ECCENTRICITY IS:

$$M_n = P_n e = (433.72^k)(1.684^{in})$$

$$= 730.4 \text{ k·in}$$

FOR COMPRESSION MEMBERS W/ NON-SPIRAL REINFORCEMENT: $\phi = 0.65$ — ACI 318-11 9.3.2.2

$$\phi P_n = (0.65)(433.72^k) = \underline{281.92^k > P_u = 37.4^k \quad OK!}$$

$$\phi M_n = (0.65)(730.4^{k\cdot in}) = \underline{474.76^{k\cdot in} > M_u = 63^{k\cdot in} \quad OK!}$$

5 squares per inch

Solve Blocks for compression length of Stem walls

Reference

Reference text

See calculations for 7" stem wall between mechanisms:

Solver Constraints Guess Values

$c := 0$ initial guess

$0 = 5.3965\, c^3 - 23.06\, c^2 + 189.9\, c - 1508.91$

find $c = 6.07$

See calculations for 7" stem wall at outer edges of box:

$c := 0$ initial guess

$0 = 16.41\, c^3 - 70.13\, c^2 + 51.09\, c - 471.98$

find $c = 4.85$

Reference

Tire Contact Area Stopping Force on Stem Wall based on Kinetic Energy and Impulse Momentum Principles

The stem wall design will be checked for a conservative force generated by the tire area rolling through the rumble edge and generating a lateral force calculated using kinetic energy and momentum-impulse theorem equations.

Stopping Force based on Standard Deceleration Rates for 32 kip Axle (16 kip tire area) using kinematic equations

$V_i := 50\ mph$ $\quad V_i = 73.3\ \frac{ft}{s}$ $\qquad W_{tire} := 16\ kip$ weight of tire contact area

$V_f := 25\ mph$ $\quad V_f = 36.7\ \frac{ft}{s}$ $\qquad g = 32.2\ \frac{ft}{s^2}$ acceleration of gravity

a_{design} is the preferred deceleration rate for 90% of all drivers and used for determining required stopping sight distances

a_{max} is the deceleration rate chosen by most drivers when confronted with the need to stop for an unexpected object

$a_{design} := -3.4\ \frac{m}{s^2}$ $\qquad a_{max} := -5.6\ \frac{m}{s^2}$

NCHRP Report 400 Chapter 4 Design Deceleration Rates

$$\Delta x_{design} := \frac{V_f^2 - V_i^2}{2 \cdot a_{design}} = 181\ ft \qquad \Delta x_{max} := \frac{V_f^2 - V_i^2}{2 \cdot a_{max}} = 110\ ft$$

distances to achieve, V_f

$$KE_i := 0.5 \cdot \left(\frac{W_{tire}}{g}\right) \cdot V_i^2 = 1337\ ft \cdot kip$$

initial vehicle kinetic energy

$$KE_f := 0.5 \cdot \left(\frac{W_{tire}}{g}\right) \cdot V_f^2 = 334\ ft \cdot kip$$

final vehicle kinetic energy

$$\Delta t_{design} := \frac{V_f - V_i}{a_{design}} = 3.29\ s \qquad \Delta t_{max} := \frac{V_f - V_i}{a_{max}} = 2\ s$$

time to achieve, V_f

$$F_{stop.design} := \left(\frac{W_{tire}}{g}\right) \cdot a_{design} = -5.55\ kip$$

Deceleration stopping force for a_{design}

$$F_{stop.max} := \left(\frac{W_{tire}}{g}\right) \cdot a_{max} = -9.14\ kip$$

Deceleration stopping force for a_{max}

Reference

Impulse Force based on Change of Momentum for 32 kip Axle (16 kip tire area) using Momentum-Impulse Theorem
The Momentum-Impulse Theorem states that the change in momentum of an object is equal to the impulse exerted on it:

Change in momentum = Impulse
P_f - P_i = Force * Time

$W_{tire} = 16\ kip$

$V_i := 50\ mph$ $V_f := 25\ mph$

initial vehicle momentum $P_i := V_i \cdot \left(\frac{W_{tire}}{g}\right) = 36\ kip \cdot s$

final vehicle momentum $P_f := V_f \cdot \left(\frac{W_{tire}}{g}\right) = 18\ kip \cdot s$

Impulse Force Magnitude based on: $F_{impulse.max} := \frac{P_f - P_i}{\Delta t_{max}} = -9.14\ kip$

$\Delta t_{max} = 2\ s$

Impulse Force Magnitude based on: $F_{impulse.design} := \frac{P_f - P_i}{\Delta t_{design}} = -5.55\ kip$

$\Delta t_{design} = 3.29\ s$

Both calculation methods confirm that under the deceleration rate a_{max}, the 16 kip tire contact area would need an opposing force of 9.14 kips applied over the period of Δt_{max}. That deceleration impulse force would be applied over the full stopping distance of Δx_{max}, but concervatively a 20" wide strip (matching width of tire contact area) of the stem wall will be checked if it can withstand this demand in shear.

$F_{impulse} := \max\left(\left|F_{impulse.max}\right|, \left|F_{impulse.design}\right|\right) = 9.137\ kip$

$V_u := 1.00\ F_{impulse} = 9.1\ kip$

The strength level shear demand due to impulse momentum since beyond the conventional AASHTO code will be factored with 1.00 as woudl be applicable to vehicular collision (VC) or friction load (FR) in AASHTO 2012 Table 3.4.1-1

The impulse force would concurently apply a flexural demand on the stem wall equivalent to the impulse force **x** the height of the stem wall ($h_{stem.wall}$) :

$h_{stem.wall} := 9\ in$ height of the stem wall

$M_{impulse} := F_{impulse} \cdot 9\ in = 82.2\ kip \cdot in$ flexural demand on stem wall due to impuse force.

$M_u := 1.00 \cdot M_{impulse} = 82.2\ kip \cdot in$

The strength level flexural demand due to impulse momentum since beyond the conventional AASHTO code loadins, will be factored with 1.00 as woudl be applicable to vehicular collision (VC) or friction load (FR) in AASHTO 2012 Table 3.4.1-1

Reference

Shear capacity of 7" thick stem wall w/ 1/8" PL on ea side (on one side for edge condition).

$\phi_v := 0.75$	shear strength reduction factor	ACI 318-11 9.3.2.3
$\lambda := 1.0$	lightweight concrete modification factor [1.0] for normalweight concrete	8.6.1
$d := 3.5\ in$	d = effective depth - distance from extreme compression fiber to centroid of tensile reinf. *for the outside edge stem walls there is 1/8" PL on only one side so the effective depth is to the center reinforcement at half the stem wall thickness*	
$b_w := 20\ in$	design strip width of stem wall	
$f'_c := 4500\ psi$	specified compressive strength of concrete	11.2.2.1
$f_y := 60\ ksi$	specified yield strength of reinforcement (psi)	
$A_s := 1.55\ in^2$	tensile reinforcement for a 20" design strip using #5 (0.31 in^2/bar) @ 4" OC	

$$\rho_w := \frac{A_s}{b_w \cdot d} = 0.0221$$ ratio of reinforcement area A_s to $b_w \cdot d$

Per ACI 318-11 Section 11.2.2.1 Eq. (11-5), shear strength of members subject to shear and flexure only

$$\phi V_{c\_11.2.2.1} := \left(1.9 \cdot \lambda \cdot \sqrt{\frac{f'_c}{psi}} + 2500 \cdot \rho_w \cdot \frac{V_u \cdot d}{M_u}\right) \cdot \frac{b_w}{in} \cdot \frac{d}{in} \cdot lbf$$

$V_u = 9.1\ kip$

$\phi V_{c\_11.2.2.1} = 10.4\ kip$

$$check.\phi V_c := \text{if } \phi V_{c\_11.2.2.1} > V_u \;\; \text{"OK"} \;\; \text{else} \;\; \text{"Shear Strength Exceeded"}$$

$check.\phi V_c = \text{"OK"}$

Reference

Flexural capacity of 7" thick stem wall w/ 1/8" PL on one side and #5@4" OC tensile reinf in middle

ACI 318-11 9.3.2.1

$\phi_b := 0.90$ flexure strength reduction factor for tension controlled sections

$$a := \frac{A_s \cdot f_y}{0.85 \cdot f'_c \cdot b_w} = 1.216\ in$$

depth of equivalent rectangular compression stress block

$$\beta_1 := \begin{cases} 0.85 & \text{if } f'_c \le 4000\ psi \\ \max\left(0.85 - 0.05 \cdot \left(\frac{f'_c - 4000\ psi}{1000\ psi}\right), 0.65\right) & \text{if } f'_c > 4000\ psi \end{cases} = 0.8$$

ratio of the depth of rectangular stress distribution to the depth of the neutral axis

$$x_b := \frac{a}{\beta_1} = 1.5\ in$$

the neutral axis is located x_b from the point of maximum compression

$$\varepsilon_t := \left(\frac{d - x_b}{x_b}\right) \cdot 0.003 = 0.0041$$

net strain in the tension steel

$$check.\varepsilon_s := \begin{cases} \text{“Compression Controlled”} & \text{if } \varepsilon_t \le 0.002 \\ \text{“Transion Range”} & \text{if } 0.002 < \varepsilon_t < 0.005 \\ \text{“Tension Controlled”} & \text{if } \varepsilon_t \ge 0.005 \end{cases}$$

$$\phi_{b.adj} := \begin{cases} 0.65 & \text{if } \varepsilon_t \le 0.002 \\ 0.65 + (\varepsilon_t - 0.002) \cdot \left(\frac{250}{3}\right) & \text{if } 0.002 < \varepsilon_t < 0.005 \\ 0.90 & \text{if } \varepsilon_t \ge 0.005 \end{cases}$$

ACI 318-11 9.3.2 Fig. R9.3.2

$check.\varepsilon_s = \text{“Transion Range”}$

$\phi_{b.adj} = 0.83$

Nominal flexural strength of stem wall 20" wide design section

$$\phi M_n := \phi_{b.adj} \cdot A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right) = 222.5\ kip \cdot in$$

$M_u = 82.2\ kip \cdot in$

$\phi M_n = 18.5\ ft \cdot kip$

$$check.\phi M_n := \begin{cases} \text{“OK”} & \text{if } \phi M_n > M_u \\ \text{“Flexural Strength Exceeded”} & \text{else} \end{cases}$$

$check.\phi M_n = \text{“OK”}$

tion time for stopping sight distance situations encompasses the capabilities of most drivers (including those of older drivers). In fact, the data shows that 2.0 sec exceeds the 85th percentile SSD perception-brake reaction time for all drivers, and 2.5 sec exceeds the 90th percentile SSD perception-brake reaction time for all drivers.

Thus, the 2.5 sec value should be used for determining required stopping sight distances; however, it should be noted that at locations where stopping sight distance is not the appropriate control, different perception-reaction times may be appropriate. For example, shorter perception-brake reaction times may be appropriate for traffic signal design where change intervals are expected, and longer perception-brake reaction times may be appropriate for intersection or interchange design where driver speed and path corrections are unexpected.

Design Deceleration

This research and other studies documented in the literature show that most drivers choose decelerations greater than 5.6 m/sec^2 when confronted with the need to stop for an unexpected object in the roadway. Approximately 90 percent of all drivers choose decelerations that are greater than 3.4 m/sec^2. These decelerations are within drivers' capability to stay within their lanes and maintain steering control during braking maneuvers on wet surfaces.

Thus, 3.4 m/sec^2 (a comfortable deceleration for most drivers) is recommended as the deceleration threshold for determining required stopping sight distance. Implicit in this deceleration threshold is the requirement that the vehicle braking system and pavement friction values are at least equivalent to 3.4 m/sec^2 (0.34 g). Skid data show that most wet pavement surfaces on state maintained roadways exceed this threshold. Braking data show that most vehicle braking systems can exceed the skidding friction values for the pavement.

Recommended Stopping Sight Distances

The recommended stopping sight distances for design are based on below average drivers detecting an unexpected object in the roadway and stopping a vehicle before striking the object. The recommended values are shown in Table 57. The values in the bottom five rows of the table represent those stopping sight distances beyond the driver's visual capabilities for detecting small objects (150 to 200 mm objects) during the day and large, low contrast objects at night.

For comparison purposes, AASHTO's 1994 design stopping sight distances are shown in Table 58 and Figure 19. Note that the recommended values are approximately midway between the 1994 minimum and desirable values for all initial speeds.

Eye Heights and Object Heights

This research and other studies documented in the literature show that more than **90 percent** of all passenger-car driver eye heights exceed 1,080 mm. This eye height encompasses an even larger proportion of the vehicle fleet when trucks and multipurpose vehicles are included in the population. Thus, 1,080 mm is recommended as the driver eye height for determining required stopping sight distances.

TABLE 57 Recommended stopping sight distances for design

Initial Speed (km/h)	Perception-Brake Reaction		Deceleration (m/s^2)	Braking Distance (m)	Stopping Sight Distance for Design (m)
	Time (s)	Distance (m)			
30	2.5	20.8	3.4	10.2	31.0
40	2.5	27.8	3.4	18.2	45.9
50	2.5	34.7	3.4	28.4	63.1
60	2.5	41.7	3.4	40.8	82.5
70	2.5	48.6	3.4	55.6	104.2
80	2.5	55.6	3.4	72.6	128.2
90	2.5	62.5	3.4	91.9	154.4
100	2.5	69.4	3.4	113.5	182.9
110	2.5	76.4	3.4	137.3	213.7
120	2.5	83.3	3.4	163.4	246.7

SECTION 400
EMBED ANGLE

By T. BATIKOV Date 2/6/18 Client UNLV Sheet 1 of

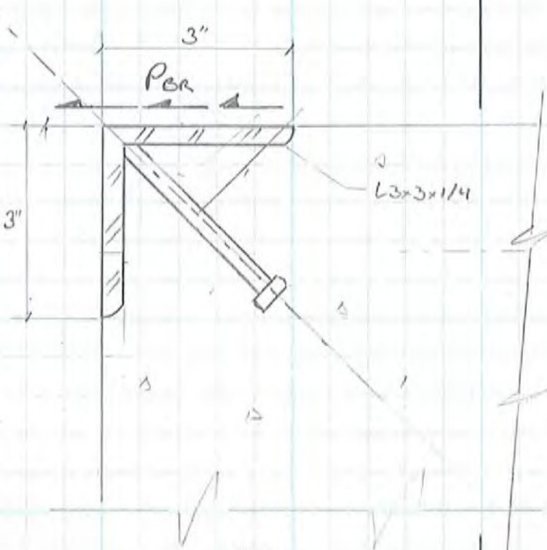
Chkd. By D. ROUNDS Description DRT RUMBLE STRIP Job No. 181307096

Design Task L3x3x1/4 EMBED ANGLE ANCHORAGE

REFERENCES

EMBED ANGLE CAPACITY TO BREAKING FORCE

THE CORNER ANGLES NEED TO BE DESIGNED TO RESIST LATERAL FORCE DUE TO DESIGN TRAFFIC LOAD BREAKING FORCE.



FIGURE

AS SHOWN ON PAGE 2, MAX P_{BR} WOULD BE 2880 PSF DUE TO 25% OF DESIGN TRUCK LL.

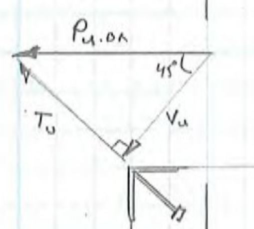
THAT LATERAL PRESSURE WILL BE FACTORED AND ONLY A PORTION OF IT APPLIED THAT IS IN CONTACT W/ HORIZONTAL LEG OF THE L3x3x1/4 ANGLES.

$$P_{U.BR} = 1.75\,(2880\,\text{PSF})\left[\frac{3''\ \text{ANGLE LEG}}{12''/\text{FT}}\right]$$
$$= 1260\ \text{PLF}$$

THE ANGLE EMBED STUDS SHALL BE DESIGNED FOR THIS FORCE. THE BRAKING FORCE CAN BE BROKEN DOWN INTO COMPONENTS REPRESENTING A TENSION AND SHEAR DEMAND ON EA STUD.

SHEAR

$$V_u = P_{U.BR}\cos(45^\circ)$$
$$= (1260\ \text{PLF})\cos(45^\circ) = 891\ \#/\text{FT}$$



TENSION

$$T_u = P_{U.BR}\sin(45^\circ)$$
$$= (1260\ \text{PLF})\sin(45^\circ) = 891\ \#/\text{FT}$$

ANCHOR ROD IN TENSION

THE REQUIRED ANCHOR AREA IN TENSION IS:

$$A_{es\,req'd} = \frac{T_u}{\phi F_{uta}}, \quad \phi = 0.75\ (\text{ACI 318-11 App D.4.3}) \quad F_{ut} = 65\ \text{KSI}\ (\text{AWS D1.1 TYPE B STUD})$$

$$= \frac{0.891\ \text{K/FT}}{(0.75)(65\ \text{KSI})} = 0.0183\ \text{in}^2$$

$$A_{es\,req'd} = 0.0205\ \text{in}^2/\text{FT}$$

TRY 3/8" ϕ STANDARD STUD, $A_{es} = \frac{\pi(0.375'')^2}{4} = 0.1104\ \text{in}^2$

$$\phi R_n = (0.75)(65\ \text{KSI})(0.1104\ \text{in}^2) = 5.382^K > 0.891\ \text{K/FT}$$

SEE MATHCAD FOR REST OF CALCULATIONS..

ACI 318-11 App. D.4.3 D.5.1.2 (D-2)

AWS D1.1 CHAPTER 7 GRADE B STUD TABLE 7.1

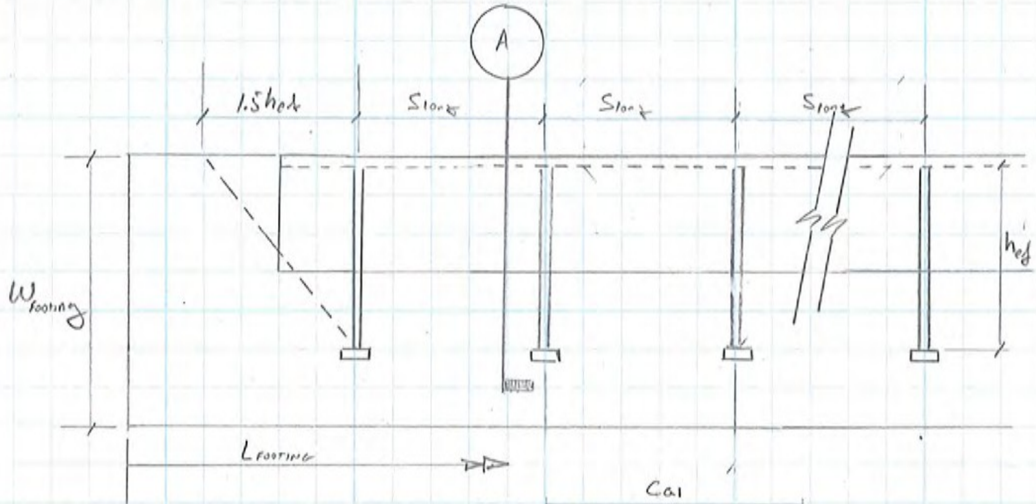
5 squares per inch

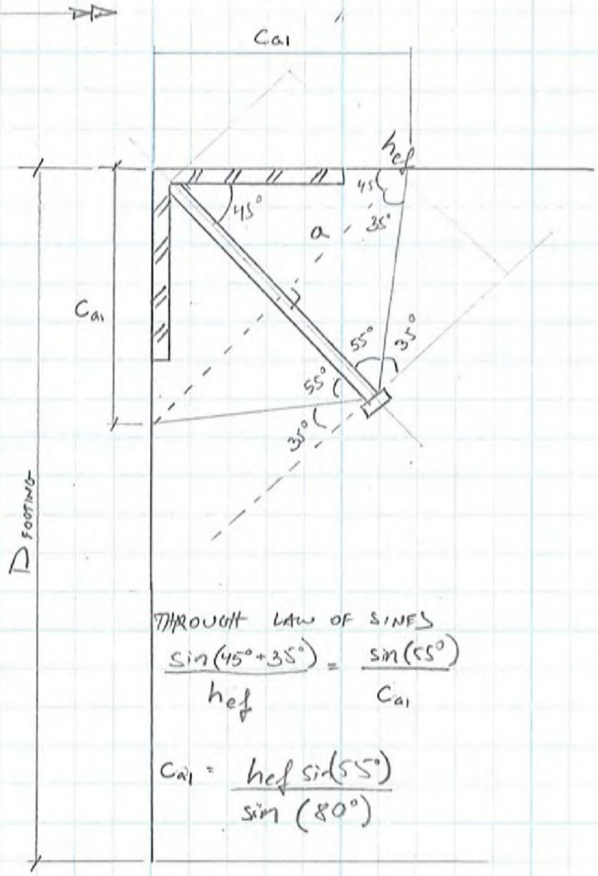
By I. BATILOV Date 2/5/18 Client UNLV Sheet 2 of

Chkd. By D. ROUNDS Description DRT RUMBLE STRIP Job No. 181307096

Design Task L3x3x1/4 EMBED ANGLE ANCHORAGE

REFERENCES





THROUGH LAW OF SINES

$$\frac{\sin(45^\circ + 35^\circ)}{h_{ef}} = \frac{\sin(55^\circ)}{C_{a1}}$$

$$C_{a1} = \frac{h_{ef}\sin(55^\circ)}{\sin(80^\circ)}$$

5 squares per inch

Conection of Embed Angle w/ Standard Studs

Reference

The rumble strip embed angle will be anchored to the concrete stem walls in between each mechanism w/ standard studs per AWS D1.1 Chapter 7. The stud size, spacing and embed depth will be dependent on the AASHTO LRFD factored braking force calculated separately. The braking force is resolved in a tension and shear force for the anchors since they are welded at a 45 degree angle to the horizontal.

ACI 318-11
Appendix D
2.1 definitions
D.4.1

Refer to hand calculations for braking force

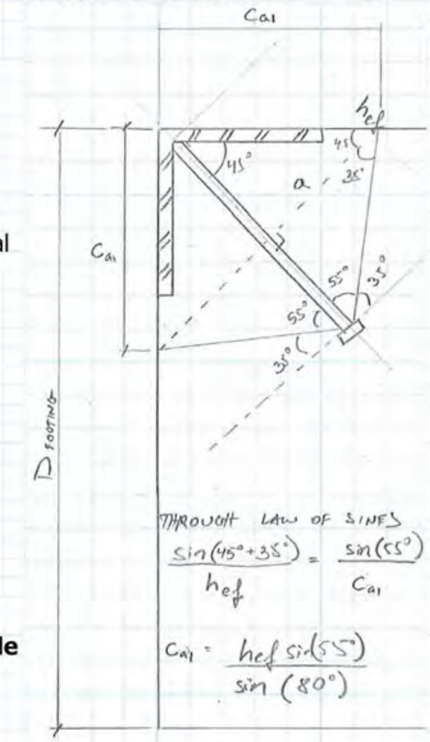
Connection Demand

$\theta_{stud} := 45\ °$ Angle of stud to vector of braking force

$P_{u.BR} := 1260\ plf$ Axial force in brace based on governing RAM seismic LC with a ρ factor of 1.0 and a Ω_o of 2.0 for Steel SCBF

Material Strengths

$f'_c := 4500\ psi$ $E_{steel} := 29000\ ksi$



Maximum Shear and Tension Forces from RAM

Following at the maximum shear and tension forces a braced frame column connection from RAM steel lateral analysis of the steel frame building:

$$N_{u\_MAX} := P_{u.BR} \cdot \cos(\theta_{stud}) = 890.955\ plf$$

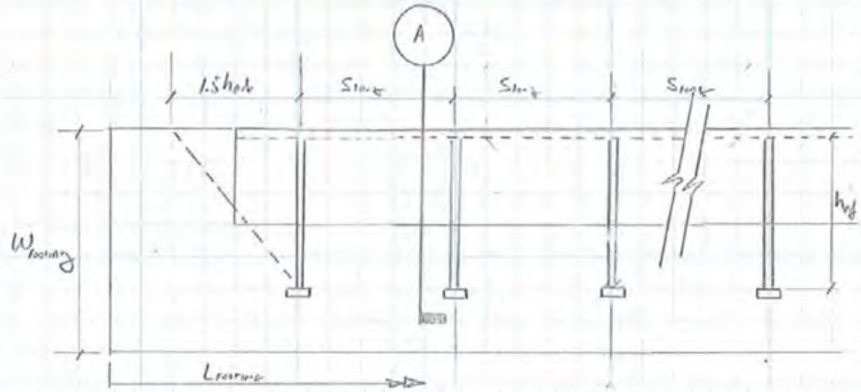
$$V_{u\_MAX} := P_{u.BR} \cdot \sin(\theta_{stud}) = 890.955\ plf$$

$N_{u\_MAX} = 891\ plf$ Maximum Tension

$V_{u\_MAX} = 891\ plf$ Maximum Shear

Dimensions of Concrete Member for Embed Angle

$W_{footing} := 7\ in$ $L_{footing} := 12\ ft$ $D_{footing} := 15\ in$



Reference

Anchor Rod / Stud Spacing

$S_{trans} := 0\ in$ $\quad S_{long} := 12\ in$ Anchor Spacing on embed angle

$L_{angle} := 11\ ft$ Length of Base PL/Angle

$W_{angle} := 3\ in$ Width of angle leg

$t_{angle} := 0.25\ in$ Thickness of Embed Angle

$Anchor_{edgeD} := 2\ in$ Edge Distance of Anchors embed angle

$d_{anchor} := \frac{3}{8}\ in$ Anchor/Stud Rod Diameter

$A_{se.N} := \frac{\pi \cdot d_{anchor}^{2}}{4} = 0.11\ in^2$ Cross-Sectional Area per Anchor Stud

$H_{stud.dia}(d_{anchor}) = 0.75\ in$ Stud Head Diameter per AWS D1.1 Figure 7.1

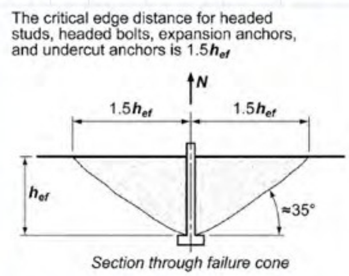
AWS D1.1 Stud Standard Dimensions per Figure 7.1

$$H_{stud.dia}(x) := \begin{cases} 0.75\ in & \text{if } x = 0.375\ in \\ 1\ in & \text{if } x = 0.50\ in \\ 1.25\ in & \text{if } x = 0.625\ in \\ 1.25\ in & \text{if } x = 0.75\ in \\ 1.375\ in & \text{if } x = 0.875\ in \\ 1.625\ in & \text{if } x = 1\ in \end{cases}$$

$e'_N := 0\ in$ eccentricity of load from centroid of anchors loaded in tension (confirm with HILTI)

$h_{ef} := 4\ in$ Define Effective Embed Depth

$\theta_{cb} := 35\ °$ Angle of failure for concrete breakout (per ACI 318-11 Figure RD.5.2.1)



ACI 318-11 Appendix D 2.1 definitions D.3.3.4.4 (d) D.4.1

Distance from center of anchor to edge of concrete perpendicular to the edge in the vertical or horizontal direction, solved via the law of sines (see figure)

$$c_{a1} := \frac{h_{ef} \cdot \sin(90\ ° - \theta_{cb})}{\sin((90\ ° - \theta_{stud}) + \theta_{cb})} = 3.327\ in$$

Distance from center of edge anchors to edge of concrete in the direction perp to c_{a1}, but parallel to embed angle:

$$c_{a2} := min(0.5 \cdot (L_{footing} - L_{angle}) + Anchor_{edgeD}, 1.5 \cdot h_{ef}) = 6\ in$$

Reference

Material Strengths

Per ACI D.3.3.7 anchor related reinforcement used in structures assigned to Seismic Design Category C, D, E, or F shall be ASTM A706 Grade 60

AISC 360 Table 2-6

$f_{y.A706.Gr60} := 60000\ psi$ $\quad f'_c := 5000\ psi$ $\quad \gamma_{conc} := 150\ pcf$

$f_{u.A706.Gr60} := 80000\ psi$ $\quad E_{steel} := 29000\ ksi$

$f_{y.stud} := 51\ ksi$ $\quad f_{u.stud} := 65\ ksi$

Stud yield and tensile strenght per AWS D1.1 Table 7.1 Type B

CHECK 1: D.3.3.4.4(a) Minumum required area of anchors based on tensile force:

Nominal Strength of the rebar Anchor is:

ACI 318-11 Appendix D D.3.3.4.4(a) D.5.1.2 (D-2)

$$1.9 \cdot f_{y.stud} = 96.9\ ksi$$

$$N_{sa} := min\left(f_{u.stud}, 1.9 \cdot f_{y.stud}, 125\ ksi\right) \cdot A_{se.N} = 7.179\ kip$$

$$\phi N_{sa} := 0.75 \cdot min\left(f_{u.stud}, 1.9 \cdot f_{y.stud}, 125\ ksi\right) \cdot A_{se.N} = 5.384\ kip$$

Required # of Anchors based on selected size:
(note "ceil(x,z)" rounds **x** up to the nearest full bar size thats also a multiple of **z**)

$$\#ofAnchors_{REQ'D} := \text{Ceil}\left(\frac{N_{u\_MAX} \cdot L_{angle}}{\phi N_{sa}}, 1\right) \qquad \frac{N_{u\_MAX} \cdot L_{angle}}{\phi N_{sa}} = 1.82$$

$$\#ofAnchors_{REQ'D} = 2 \qquad \#ofAnchors := \text{Floor}\left(\frac{L_{angle}}{S_{long}}, 1\right) = 11$$

\# of anchors provided

$$\phi N_{sa.g} := \phi N_{sa} \cdot \#ofAnchors = 59.23\ kip \qquad N_{u\_MAX} \cdot L_{angle} = 9.8\ kip$$

$CHECK\_\phi N_{sa.g} :=$ if $\phi N_{sa.g} > N_{u\_MAX} \cdot L_{angle}$
‖ "OK"
else
‖ "Size & Spacing Anchors NOT OK"

$CHECK\_\phi N_{sa.g} =$ "OK"

CHECK 2: D.3.3.4.4(b) Concrete breakout strength per ACI 318-11 Appendix D using the Concrete Capacity Design (CCD) method

ACI 318-11 Appendix D D.3.3.4.4(b)

$e'_N = 0\ in$ $\quad D_{footing} - 3\ in = 12\ in$

eccentricity of load from centroid of anchors loaded in tension (confirm with HILTI)

HILTI

$h_{ef} = 4\ in$

Define Effective Embed Depth

D.5.2 D.5.2.4 fig. RD.5.2.4

$$\Psi_{ec.N} := min\left(1, \frac{1}{\left(1 + \frac{2 \cdot e'_N}{3 \cdot h_{ef}}\right)}\right) = 1$$

Modification factor for anchor groups loaded eccentrically in tension (Eq D-8)

$$1.5 \cdot h_{ef} = 6\ in$$

$$c_{a.MIN} := min\left(c_{a1}, c_{a2}\right) = 3.327\ in$$

minumum distance from center of anchor to edge of concrete

Reference

$$\Psi_{ed.N} := \begin{cases} 1.0 & \text{if } c_{a.MIN} \geq 1.5 \cdot h_{ef} \\ 0.7 + 0.3 \cdot \dfrac{c_{a.MIN}}{1.5 \cdot h_{ef}} & \text{if } c_{a.MIN} < 1.5 \cdot h_{ef} \end{cases}$$

Modification factor for edge effects for single anchors or anchor group loaded in tension (D-9) (D-10) — ACI 318-11 Appendix D D.5.2.5

$\Psi_{ed.N} = 0.87$

Modification factor for cast-in or post-installed anchors locate in a cracked or non cracked region of a concrete members (D.5.2.6) — D.5.2.6

$\Psi_{c.N} := 1.00$

[**1.25**] for cast-in anchors/no cracking
[**1.40**] for post-installed anchors/no cracking
k.c is 17
[**1.00**] for all anchors w/ cracked concrete

Second Modification factor for cast-in or post-installed anchors located in a cracked or non cracked region of a concrete members (D.5.2.7) — ACI 318-11 Appendix D D.5.2.7

$\Psi_{cp.N} := 1.00$

Refer to code for post-installed anchors
[**1.00**] for all cast in anchors

The basic concrete breakout strength of a single anchor in tension in cracked concrete N_b per D5.2.2 shall be found as follows — D.5.2.2

$k_c := 24$

[**24**] for cast-in anchors
[**17**] for post-installed anchors

$\lambda := 1.00$

Lightweight concrete modification factor — ACI 318-11 8.6.1
[**1.00**] for normal weight concrete
[**0.85**] for sand-lightweight concrete
[**0.75**] for all lightweight concrete

$\lambda_a := 1.0 \cdot \lambda$

$\lambda_a = 1.00$

Per D.3.6 Modification factor for failure of anchors in Lightweight concrete — D.3.6
[**1.0**λ] for cast-in and undercut anchor concrete failure
[**0.8**λ] for expansion/adhesive anchor concrete failure
[**0.7**λ] for adhesive anchor bond failure per (D-22) — ACI 318-11 Appendix D

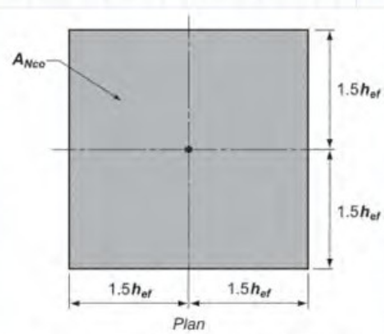
$$N_{b.1} := k_c \cdot \lambda_a \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{1.5} \cdot lbf = 13.6\ kip \qquad \text{(D-6)}$$

The basic concrete breakout strength of a single anchor in tension in cracked concrete

$$N_{b.2} := 16 \cdot \lambda_a \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{h_{ef}}{in}\right)^{\left(\frac{5}{3}\right)} \cdot lbf = 11.4\ kip \qquad \text{(D-7)}$$

$$N_b := \begin{cases} N_{b.2} & \text{if } 11\ in \leq h_{ef} \leq 25\ in \\ N_{b.1} & \text{else} \end{cases}$$

$N_b = 13.576\ kip$



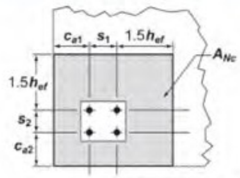
$A_{Nco} = (2 \times 1.5h_{ef}) \times (2 \times 1.5h_{ef}) = 9h_{ef}^2$

Breakout area for a single anchor not limited by edge distance or spacing of anchors (D-5)

$A_{Nco} := 9 \cdot h_{ef}^2 = 1\ ft^2 \qquad A_{Nco} = 144\ in^2$

Reference

Breakout area for anchor group limited by edge distance



$A_{Nc} = (c_{a1} + s_1 + 1.5h_{ef})(c_{a2} + s_2 + 1.5h_{ef})$
if c_{a1} and $c_{a2} < 1.5h_{ef}$
and s_1 and $s_2 < 3h_{ef}$

$Rows_{bar} := 1$ — Define the # of rows the anchor group is laid out in longitudinally — ACI 318-11 Appendix D 2.1 definitions Fig. RD.5.2.1

$S_{trans} = 0\ in$ — rebar anchor spacing as defined earlier

$S_{long} = 12\ in$

$\frac{\#ofAnchors}{Rows_{bar}} = 11$ anchors/row

Distance from center of anchors to edge of concrete in the direction of shear

$c_{a1} = 3.327\ in$ $\qquad c_{a1} = 0.277\ ft$

Distance from center of anchors to edge of concrete in the direction perpendicular to c_{a1}

$c_{a2} = 6\ in$

$c_{a.MIN} = 3.327\ in \qquad 1.5 \cdot h_{ef} = 6\ in$

$min(1.5 \cdot h_{ef}, c_{a1}) = 3.33\ in$

$min(1.5 \cdot h_{ef}, c_{a2}) = 6\ in$

$$A_{Nc} := \left((2 \cdot min(1.5 \cdot h_{ef}, c_{a2})) + \left(\frac{\#ofAnchors}{Rows_{bar}} - 1\right) \cdot S_{long}\right)((2 \cdot min(1.5 \cdot h_{ef}, c_{a1})) + (Rows_{bar} - 1) \cdot S_{trans})$$

$A_{Nc} = 6.1\ ft^2$

$A_{Nc} = 878.4\ in^2$

Breakout area for anchor group limited by edge distances of spread mat footing

Per D.5.2.1, breakout area for anchor group limited by edge distances of spread mat footing, A_{Nc} shall not exceed $n \cdot A_{Nco}$ where n is the # of anchors

$A_{Nc.MAX} := \#ofAnchors \cdot A_{Nco} = 11\ ft^2$

```
CHECK_A_Nco := if A_Nc < A_Nc.MAX
                 ‖ "OK"
               else
                 ‖ "CHECK ANCHOR SPACING"
```

$CHECK\_A_{Nco} = \text{"OK"}$

$$N_{cbg} := \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec.N} \cdot \Psi_{ed.N} \cdot \Psi_{c.N} \cdot \Psi_{cp.N} \cdot N_b$$

Nominal concrete breakout strength in tension of a group of anchors — ACI 318-11 Appendix D (D-4)

$N_{cbg} = 71.746\ kip$

$\phi_{concrete} := 0.70$

Refer to ACI 318-11 D.4.3(c)
ϕ =0.75 for Condition A
(where supplementary reinforcement is provided)
ϕ =0.70 for Condition B
(where supplementary reinforcement is NOT provided)

ACI 318-11

D.4.3(c)
D.3.3.4.4 (b)

Reference

$\phi N_{cbg} := \phi_{concrete} \cdot N_{cbg} = 50.222\ kip$

$Utilization_{BREAKOUT} := \dfrac{N_{u\_MAX} \cdot L_{angle}}{\phi N_{cbg}} = 0.195$ $\qquad N_{u\_MAX} \cdot L_{angle} = 9.8\ kip$

$CHECK\_\phi N_{cbg} :=$ if $Utilization_{BREAKOUT} < 1.0$
‖ "OK"
else
‖ " Increase h.ef, or add anchor reinforcement per D.5.2.9"

$CHECK\_\phi N_{cbg} =$ "OK"

CHECK 3: Concrete Pullout Strength 0.75 ϕN_{pn} per ACI 318-11 App. D D.3.3.4.4 (c) for a single anchor, or the most highly stressed anchor in a group of anchors.

The A_{brg} area is calculated based on the head diameter of standard studs per AWS D1.1 Figure 7.1.

ACI 318-11 D.3.3.4.4 (c) D.5.3

$H_{stud.dia}\left(d_{anchor}\right) = 0.75\ in$ Stud Head Diameter per AWS D1.1 Figure 7.1

$A_{se.N} = 0.11\ in^2$

$A_{brg} := \dfrac{\pi \cdot \left(H_{stud.dia}\left(d_{anchor}\right)\right)^2}{4} - A_{se.N} = 0.331\ in^2$ Net bearing area of stud head

Per D.5.3.6 modification factor for cracked or uncracked concrete:

ACI 318-11 D.5.3.6

$\psi_{c.P} := 1.0$

[**1.4**] for no cracking of concrete at service level loads
[**1.0**] for cracking of concrete at service level loads

Per D.5.3.4 pullout strength in tension of a single headed stud/headed bolt, N_p shall not exceed:

ACI 318-11 D.5.3.4 (D-14)

$N_p := 8 \cdot A_{brg} \cdot f'_c = 13.254\ kip$

Per D.5.3.1 nominal pullout strength in tension of a single anchor shall not exceed:

D.5.3.1 (D-13)

$N_{pn} := \psi_{c.P} \cdot N_p = 13.254\ kip$

Per D.4.3, anchor governed by concrete breakout, side-face blowout, pullout, or pryout shall use reduction factor ϕ below:

ACI 318-11 D.4.3 Fig RD.5.2.9

$\phi_{conc.Tension} := 0.70$

[**0.75**] if supplementary reinforcement for TENSION is included (Condition A) similar to Fig RD.5.2.9
[**0.70**] if no supplementary reinforcement for TENSION is included (Condition B) similar to Fig RD.5.2.9

Reference

Concrete Pullout Strength 0.75 ϕN_{pn} of an individual anchor in a group needs to exceed max tensile load on an individual anchor, $N_{ua.i}$ which is identified through HILTI modelling of the anchor layout or defined here:

$$N_{ua.i} := \frac{\left(N_{u\_MAX} \cdot L_{angle}\right)}{\#ofAnchors} = 891 \ \mathbf{lbf}$$

$$\phi N_{pn} := 0.75 \cdot \phi_{conc.Tension} \cdot N_{pn} = 6958.1 \ \mathbf{lbf}$$

$$CHECK\_\phi N_{pn} := \text{if } \phi N_{pn} > N_{ua.i}$$

"Concrete Pullout OK"

else

"Revise Stud Size or Spacing"

$$CHECK\_\phi N_{pn} = \text{"Concrete Pullout OK"}$$

Reference

CHECK 4: Concrete Side-Face Blowout Strength $0.75\ \phi N_{sb}$ for single anchor and for group $0.75\ \phi N_{sbg}$ per ACI 318-11 Appendix D, D.3.3.4.4 (d)

ACI 318-11 D.3.3.4.4 (d) D.5.4

$h_{ef} = 4\ in$ $\quad 2.5 \cdot c_{a1} = 8.318\ in$ $\quad s := S_{long} = 12\ in$ distance between the outer anchors along the edge where break

$c_{a1} = 3.327\ in$ $\quad 3.0 \cdot c_{a1} = 9.981\ in$ $\quad \dfrac{c_{a2}}{c_{a1}} = 1.803$

$c_{a2} = 6\ in$

For single anchor with deep embedment close to an edge

ACI 318-11 Appendix D D.5.4.1 (D-16)

$$N_{sb} := \begin{cases} \text{if } h_{ef} > 2.5 \cdot c_{a1} \\ \quad \text{if } c_{a2} \ge 3 \cdot c_{a1} \\ \quad\quad \left(160 \cdot c_{a1} \cdot \sqrt{\dfrac{A_{brg}}{in^2}}\right) \cdot \lambda_a \cdot \sqrt{\dfrac{f'_c}{psi}} \cdot lbf \\ \quad \text{if } c_{a2} < 3 \cdot c_{a1} \\ \quad\quad \text{if } 1 \le \dfrac{c_{a2}}{c_{a1}} \le 3 \\ \quad\quad\quad \left(1 + \dfrac{c_{a2}}{c_{a1}}\right) \cdot \dfrac{1}{4} \cdot \left(\left(160 \cdot c_{a1} \cdot \sqrt{\dfrac{A_{brg}}{in^2}}\right) \cdot \lambda_a \cdot \sqrt{\dfrac{f'_c}{psi}}\right) \cdot lbf \\ \text{else} \\ \quad \text{"NO RISK OF SIDE-FACE BLOW OUT"} \end{cases}$$

$N_{sb} =$ "NO RISK OF SIDE-FACE BLOW OUT"

For multiple anchors with deep embedment close to an edge

ACI 318-11 Appendix D D.5.4.2 (D-17)

$$N_{sbg} := \begin{cases} \text{if } h_{ef} > 2.5 \cdot c_{a1} \\ \quad \text{if } s < 6 \cdot c_{a1} \\ \quad\quad \left(1 + \dfrac{s}{6 \cdot c_{a1}}\right) \cdot N_{sb} \\ \text{else} \\ \quad \text{"NO RISK OF SIDE-FACE BLOW OUT"} \end{cases} \qquad 6 \cdot c_{a1} = 19.963\ in$$

$N_{sbg} =$ "NO RISK OF SIDE-FACE BLOW OUT"

$$\phi N_{sbg} := \begin{cases} \text{if } h_{ef} > 2.5 \cdot c_{a1} \\ \quad \phi_{conc.Tension} \cdot N_{sbg} \\ \text{else} \\ \quad \text{"NO RISK OF SIDE-FACE BLOW OUT"} \end{cases} = \text{"NO RISK OF SIDE-FACE BLOW OUT"}$$

$\phi N_{sbg} =$ "NO RISK OF SIDE-FACE BLOW OUT"

$CHECK\_\phi N_{sbg} =$ "NG or NO CALC"

Reference

CHECK 5: Bond strength of an adhesive anchor ϕN_a for single anchor and for group ϕN_{ag} per ACI 318-11 Appendix D, D.3.3.4.4 (e) **NOT required for** cast-in headed anchors

CHECK 6: Steel Strength of Anchor in Shear ϕV_{sa} per ACI 318-11 Appendix D (D.6.1)

ACI 318-11
Appendix D
D.3.3.5.2
D.3.3.5.3(c)
D.4.1
Table D.4.1.1

The anchor or group of anchors are designed for the maximum shear obtained from design load combinations that include Ω_o*E

$$A_{se.N} = 0.11\ in^2$$

$$min\left(f_{u.stud}, 1.9 \cdot f_{y.stud}, 125\ ksi\right) = 65\ ksi$$

D.6.1.2
(D-29)

$$V_{sa} := 0.60 \cdot min\left(f_{u.stud}, 1.9 \cdot f_{y.stud}, 125\ ksi\right) \cdot A_{se.N}$$

nominal shear strength per stud anchor

$$V_{sa} = 4.307\ kip$$

$\phi_v := 0.65$ per D.4.3 for shear when anchor governed by ductile steel element

ACI 318-11
Appendix D
D.4.3

$\phi_{GROUT} := 1.00$

Per D.6.1.3
[**1.00**] for anchors with **no** built-up grout pad
[**0.80**] for anchors with built-up grout pad

$$\phi V_{sa} := \phi_v \cdot \phi_{GROUT} \cdot V_{sa} = 2800\ lbf \qquad V_{ua.i} := \frac{V_{u\_MAX} \cdot L_{angle}}{\#ofAnchors} = 0.891\ kip$$

```
CHECK_ϕVsa := if ϕVsa > Vua.i
                 ‖ "OK"
              else
                 ‖ "ADD MORE ANCHORS or RESIZE"
```

$CHECK\_\phi V_{sa} =$ "OK"

CHECK 7: Concrete Breakout Strength ϕV_{cb} in Shear per ACI 318-11 Appendix D (D.6.2)

For shear force perpendicular to the edge on a single anchor:

$$c_{a1} = 3.327\ in$$
$$c_{a2} = 6\ in$$

Recall distance from center of anchors to edge of concrete in the direction of shear and that perpendicular.

$$1.5 \cdot c_{a1} = 4.991\ in$$

$$h_a := D_{footing} = 15\ in$$

$$c'_{a1} := \begin{cases} \max\left(\dfrac{c_{a2}}{1.5}, \dfrac{h_a}{1.5}, \dfrac{s}{3}\right) & \text{if } \max\left(c_{a2}, h_a\right) < 1.5 \cdot c_{a1} \\ c_{a1} & \text{else} \end{cases}$$

Per ACI D.6.2.4, where anchors are located in narrow sections of limited thickness so that both edge distances c_{a2} and h_a are less than 1.5 c_{a1} the value of c_{a1} used for calculating A_{Vc} as well as in eqns (D-32) through (D-39) shall not exceed largest of:

a) $c_{a2}/1.5$, where c_{a2} is largest edge distance
b) $h_a/1.5$; and
c) s/3, where s is the max spacing perpendicular to direction of shear, between anchors in group

ACI 318-11
Appendix D
D.6.2
D.6.2.1
D.6.2.4

$$c'_{a1} = 3.327\ in$$

Reference

$A_{Vco} := 4.5 \cdot (c'_{a1})^2 = 49.8\ in^2$

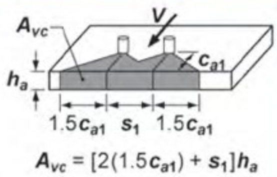
Projected concrete failure area of a single anchor for calculating shear strength if not limited by corner influences, spacing, or member thickness (D-32)

Actual projected concrete failure area of a group of anchors for calculating shear strength if when limited by corner influences, spacing, or member thicknesses

ACI 318-11 Appendix D D.6.2 D.6.2.1 D.6.2.4

$A_{Vc} := min\left(L_{footing}, ((\#ofAnchors - 1) \cdot S_{long} + 2 \cdot (1.5 \cdot c'_{a1}))\right) \cdot min\left(D_{footing}, (1.5 \cdot c'_{a1})\right) = 4.505\ ft^2$

$A_{Vc} = 648.7\ in^2$



$min\left(D_{footing}, (1.5 \cdot c'_{a1})\right) = 4.991\ in$

$l_e := min\left(h_{ef}, 8 \cdot d_{anchor}\right) = 3\ in$

load bearing length of the anchor for shear $l_e = h_{ef}$ but no longer that $8\, d_{anchor}$

D.6.2.2

Per D.6.2.2, the basic concrete breakout strength in shear of a single anchor in cracked concrete shall be the smaller of (a) and (b)

(a) $V_{b.1} := \left(7 \cdot \left(\frac{l_e}{d_{anchor}}\right)^{0.2} \cdot \sqrt{\frac{d_{anchor}}{in}}\right) \cdot \lambda_a \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c'_{a1}}{in}\right)^{1.5} \cdot lbf = 2.788\ kip$ (D-33)

(b) $V_{b.2} := 9 \cdot \lambda_a \cdot \sqrt{\frac{f'_c}{psi}} \cdot \left(\frac{c'_{a1}}{in}\right)^{1.5} \cdot lbf = 3.862\ kip$ (D-34)

$V_b := min\left(V_{b.1}, V_{b.2}\right)$

$V_b = 2.788\ kip$

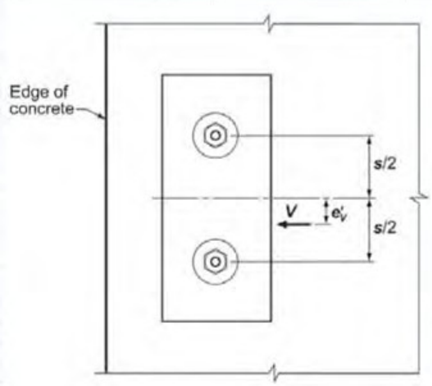
Per D.6.2.5 Modification factor for anchor groups loaded eccentrically in shear

ACI 318-11 Appendix D D.6.2.5

$e'_V := 0\ in$ eccentricity in anchors loaded in shear

$$\Psi_{ec.V} := min\left(1, \frac{1}{\left(1 + \frac{2 \cdot e'_V}{3 \cdot c'_{a1}}\right)}\right) \quad \text{(D-36)}$$

$\Psi_{ec.V} = 1.0$



Plan

Reference

Modification factor for edge effect for a singe anchor or group of anchors loaded in shear

$$\Psi_{ed.V} := \begin{cases} \text{if } c_{a2} \geq 1.5 \cdot c'_{a1} \\ \quad 1.00 \\ \text{if } c_{a2} < 1.5 \cdot c'_{a1} \\ \quad 0.7 + 0.3 \cdot \dfrac{c_{a2}}{1.5 \cdot c'_{a1}} \end{cases}$$

(D-37)
(D-38)

ACI 318-11
Appendix D
D.6.2.6

$\Psi_{ed.V} = 1.00$

Modification factor for concrete cracked state and edge reinforcement for a singe anchor or group of anchors loaded in shear

$\Psi_{c.V} := 1.0$

[**1.4**] for anchors in concrete with no cracking
[**1.0**] for anchors in cracked concrete with no supplementary reinforcement or with edge reinforcement smaller than #4 bar
[**1.2**] for anchors in cracked concrete with supplementary reinforcement or with edge reinforcement #4 bar or greater
[**1.4**] for anchors in cracked concrete supplementary reinforcement #4 bar or greater between anchor and edge, and with the reinforcement enclosed with stirrups spaced 4" or less

ACI 318-11
Appendix D
D.6.2.7

Modification factor for anchors located in a concrete member where $h_a < 1.5 \cdot c'_{a1}$

$$\Psi_{h.V} := \max\left(1, \sqrt{\frac{1.5 \cdot c'_{a1}}{h_a}}\right)$$

(D-39)

ACI 318-11
Appendix D
D.6.2.8

$\Psi_{h.V} = 1$

For shear force perpendicular to the edge on a group of anchors:

$$V_{cbg} := \frac{A_{Vc}}{A_{Vco}} \cdot \Psi_{ec.V} \cdot \Psi_{ed.V} \cdot \Psi_{c.V} \cdot \Psi_{h.V} \cdot V_b \qquad V_{cbg} = 36.309\ \mathbf{kip}$$

(D-31)

$\phi_{conc.Shear} := 0.70$

[**0.75**] if supplementary reinforcement for SHEAR is included (Condition A) similar to Fig RD.6.2.9(b)
[**0.70**] if no supplementary reinforcement for SHEAR is included (Condition B) similar to Fig RD.6.2.9(b)

ACI 318-11
Appendix D
D.4.3

$\phi V_{cbg} := \phi_{conc.Shear} \cdot V_{cbg} = 25.416\ \mathbf{kip}$

$V_{u\_MAX} \cdot L_{angle} = 9.8\ \mathbf{kip}$

$$CHECK\_\phi V_{cbg} := \begin{cases} \text{if } \phi V_{cbg} > V_{u\_MAX} \cdot L_{angle} \\ \quad \text{"OK"} \\ \text{else} \\ \quad \text{"CONCRETE BREAKOUT"} \end{cases}$$

$CHECK\_\phi V_{cbg} = \text{"OK"}$

CHECK 8: Steel Failure from Spalling Adjusted Lever Arm

Reference

Level of Restraint

The value α_M depends on the degree of restraint of the anchor at the side of the fixture of the application in question and shall be judged according to good engineering practice.

No restraint (α_M = 1.0) shall be assumed if the fixture can rotate freely.
Full restraint (α_M = 2.0) may be assumed only if the fixture cannot rotate.

$\alpha_M := 2.0$ Level of restraint

$GroutThickness := 0\ in$ Thickness of grout under base PL

$t_{angle} = 0.25\ in$ Thickness of steel base PL

$$z_{standoff} := GroutThickness + \frac{t_{angle}}{2} = 0.125\ in$$

$$L_b := z_{standoff} + 0.5 \cdot d_{anchor} = 0.313\ in$$ internal lever arm adjusted for spalling of the surface concrete

$$M_{s0} := 1.2 \cdot \left(\frac{\pi \cdot d_{anchor}^{\ 3}}{32}\right) \cdot f_{u.stud}$$ characteristic flexural resistance of anchor

$M_{s0} = 404\ in \cdot lbf$

$$1 - \frac{N_{ua.i}}{\phi N_{sa}} = 0.835$$

$$M_s := M_{s0} \cdot \left(1 - \frac{N_{ua.i}}{\phi N_{sa}}\right) = 337\ in \cdot lbf$$ Resultant Flexural Resistance of Anchor

$$V_{Ms} := \frac{\alpha_M \cdot M_s}{L_b} = 2157\ lbf$$ Bending equation for stand-off

$\phi V_{Ms} := 0.65 \cdot V_{Ms} = 1401.9\ lbf$

$V_{ua.i} = 891\ lbf$

```
CHECK_ϕV_Ms := if ϕV_Ms ≥ V_ua.i
                   ‖ "OK"
               else
                   ‖ "Revise ANCHOR Diameter"
```

$CHECK\_\phi V_{Ms} =$ "OK"

Reference

CHECK 9: Concrete Pryout Strength ϕV_{cpg} of Anchors in Shear per ACI 318-11 Appendix D (D.6.3)

For a group of anchors:

ACI 318-11 Appendix D D.6.3.1

$$k_{cp} := \begin{cases} 1.00 & \text{if } h_{ef} < 2.5\ in \\ 2.00 & \text{if } h_{ef} \geq 2.5\ in \end{cases} = 2.00$$

For cast-in, expansion, and undercut anchors, N_{cpg} shall be taken as N_{cbg} determined from eq. (D-4), and for adhesive anchors, N_{cpg} shall be lesser of N_{ag} (D-19) and N_{cbg} (D-4)

$$N_{cbg2} := \frac{A_{Nc}}{A_{Nco}} \cdot \Psi_{ec.V} \cdot \Psi_{ed.N} \cdot \Psi_{c.N} \cdot \Psi_{cp.N} \cdot N_b$$

Nominal concrete breakout strength in tension of a group of anchors, note that for shear related pry out $\Psi_{ec.N}$ is replaced with $\Psi_{ec.V}$

ACI 318-11 Appendix D (D-4)

$N_{cbg2} = 71.746\ kip$

ACI 318-11 Appendix D (D-41)

$N_{cpg} := N_{cbg2} = 71.746\ kip$

$V_{cpg} := k_{cp} \cdot N_{cpg} = 143.492\ kip$

$\phi V_{cpg} := \phi_{conc.Shear} \cdot V_{cpg} = 100.444\ kip$

$V_{u\_MAX} \cdot L_{angle} = 9.8\ kip$

$$CHECK\_\phi V_{cpg} := \begin{cases} \text{“OK”} & \text{if } \phi V_{cpg} > V_{u\_MAX} \cdot L_{angle} \\ \text{“CONCRETE PRYOUT”} & \text{else} \end{cases}$$

$CHECK\_\phi V_{cpg} = \text{“OK”}$

Reference

ACI 318-11 Appendix D D.7

CHECK 10: Interaction of Tensile and Shear Forces per ACI 318-11 Appendix D (D.7)

Steel Strength in Tension per Anchor (D.5.1)

$\phi N_{sa} = 5.4\ \text{kip}$

Max Tensile Load per Anchor

$N_{ua.i} = 0.891\ \text{kip}$

$\frac{N_{ua.i}}{\phi N_{sa}} = 0.17$

Concrete Breakout Strength in Tension (D.5.2)

$\phi N_{cbg} = 50.222\ \text{kip}$

Max Tensile Load for Anchor Group

$N_{u\_MAX} \cdot L_{angle} = 9.8\ \text{kip}$

$\frac{N_{u\_MAX} \cdot L_{angle}}{\phi N_{cbg}} = 0.2$

WHEN $\frac{N_{u\_MAX}}{\phi N_{cbg}}$ **MORE THAN 1.0 ANCHOR REINFORCEMENT IN TENSION SHALL BE PROVIDED**

Concrete Pullout Strength in Tension per Anchor (D.5.3)

$\phi N_{pn} = 6.958\ \text{kip}$

Max Tensile Load per Anchor

$N_{ua.i} = 0.891\ \text{kip}$

$\frac{N_{ua.i}}{\phi N_{pn}} = 0.13$

Steel Strength in Shear per Anchor (D.6.1)

$\phi V_{sa} = 2.8\ \text{kip}$

Max Shear Load per Anchor

$V_{ua.i} = 0.891\ \text{kip}$

$\frac{V_{ua.i}}{\phi V_{sa}} = 0.32$

Concrete Breakout Strength in Shear (D.6.2)

$\phi V_{cbg} = 25.416\ \text{kip}$

Max Tensile Load for Anchor Group

$V_{u\_MAX} \cdot L_{angle} = 9.8\ \text{kip}$

$\frac{V_{u\_MAX} \cdot L_{angle}}{\phi V_{cbg}} = 0.39$

WHEN $\frac{V_{u\_MAX}}{\phi V_{cbg}}$ **MORE THAN 1.0 ANCHOR REINFORCEMENT IN SHEAR SHALL BE PROVIDED**

Steel Failure due to Spalling Adjusted Lever Arm (HILTI Check)

$\phi V_{Ms} = 1.402\ \text{kip}$

Max Shear Load per Anchor

$V_{ua.i} = 0.891\ \text{kip}$

$\frac{V_{ua.i}}{\phi V_{Ms}} = 0.64$

Concrete Pryout Strength from Shear (D.6.3)

$\phi V_{cpg} = 100.444\ \text{kip}$

Max Tensile Load for Anchor Group

$V_{u\_MAX} \cdot L_{angle} = 9.8\ \text{kip}$

$\frac{V_{u\_MAX} \cdot L_{angle}}{\phi V_{cpg}} = 0.1$

Reference

ACI 318-11
Appendix D
D.7

governing Utilization in Tension (D.7.2)

$$\beta_{N.max} := \max\left(\frac{N_{ua.i}}{\phi N_{sa}}, \frac{N_{ua.i}}{\phi N_{pn}}\right) = 0.165$$

governing Utilization in Shear (D.7.1)

$$\beta_{V.max} := \max\left(\frac{V_{ua.i}}{\phi V_{sa}}, \frac{V_{ua.i}}{\phi V_{Ms}}, \frac{V_{u\_MAX} \cdot L_{angle}}{\phi V_{cpg}}\right) = 0.636$$

$$\beta_{N.max} + \beta_{V.max} = 0.801 \qquad \text{(D-42)}$$

```
CHECK_Interaction := | if β_V.max ≤ 0.2
                     |   | if β_N.max ≤ 1.0
                     |   |   | "OK"
                     | if β_N.max ≤ 0.2
                     |   | if β_V.max ≤ 1.0
                     |   |   | "OK"
                     | else
                     |   | if β_N.max + β_V.max ≤ 1.2
                     |   |   | "OK"
                     |   | else
                     |   |   | "ANCHOR DESIGN INSUFFICIENT"
```

CHECK_Interaction = "OK"

SECTION 500
CRANE PICK POINT
CONNECTION DESIGN

MWH now part of Stantec

By I. BATILOV Date 2/12/18 Client UNLV Sheet 1 of

Chkd. By D. ROUNDS Description DRT RUMBLE STRIP Job No. 181307096

Design Task CRANE LIFT POINT DESIGN

REFERENCES

THE RUMBLE STRIP NEEDS TO BE CRANED INTO PLACE, AND PICK POINTS NEED TO BE DESIGNED TO TAKE THE DEAD LOAD OF THE BOX.

THE BOX SELF-WT CONSERVATIVELY ASSUMING IT IS ONE SOLID CONCRETE MASS IS:

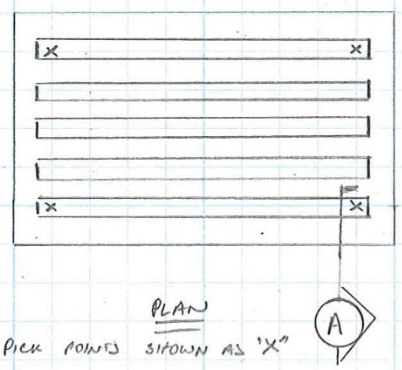
$$DL = (12\text{ FT LONG})(5.5\text{ FT WIDE})\left(\frac{9''\text{ POCKET} + 8''\text{ SLAB}}{12''/\text{FT}}\right)(\underbrace{150\text{ pcf}}_{\gamma_{conc.}})$$

$$= 14{,}025\text{ LBS}$$

GOVERNING LC FOR LIFTING SELF-WT OF BOX WILL BE ACI 318-11, LC.1 — ACI 318-11 9.2.1

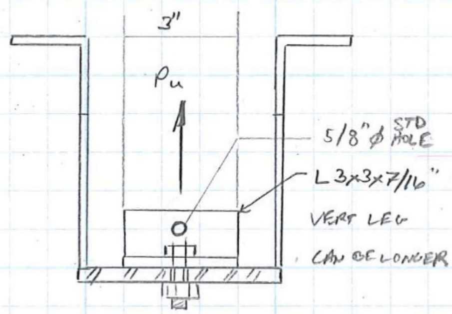
$$= 1.4DL = 1.4(14{,}025\text{ LBS}) = 19{,}635\text{ LBS}$$

DESIGN 4 PICK POINTS TO TAKE THAT FACTORED LOAD.

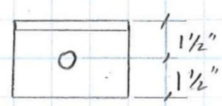


PLAN

* PICK POINTS SHOWN AS 'X'



SECTION A



PER PICK POINT

$$P_u = 19{,}635\text{ LBS}/4 = 4909\text{ LBS} = 4.91^k$$

TRY 5/8" Ø ASTM A325 BOLTS, $A_b = 0.307\text{ in}^2$ — AISC 360-10 TABLE 7-2

TENSILE STRENGTH $\phi r_n = 20.7^k > P_u$ OK, $F.S. = \dfrac{\phi r_n}{P_u} = 4.22$

SELECT ATTACHMENT ANGLE TO ELIMINATE PRYING ACTION PER AISC SECTION 9, — CHAPTER 9, Pg. 9-10

$$t_{min} = \sqrt{\frac{4Tb'}{\phi p F_u}}$$

$T = T_u = 4.91^k$

$$b' = \left(b - \frac{d_b}{2}\right) = \left(1.5'' - \frac{\overset{\text{BOLT }\phi}{5/8''}}{2}\right) = 1.1875''$$

$p = 2b = 2(1.5'') = 3''$

$F_u = 58\text{ ksi}$ FOR A36 ANGLE

5 squares per inch

REFERENCE

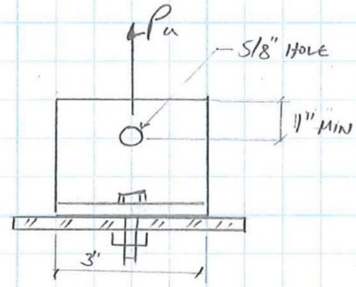
$t_{min} = \sqrt{\dfrac{4(4.91^k)(1.1875")}{(0.90)(3")(58ksi)}} = 0.386"$ MIN TO ELIMINATE PRYING ACTION

CHOOSE L3x 3x 7/16 MIN 7/16" = 0.4375" > t_{min} OK!

MIN EDGE DISTANCE FOR 5/8"Ø BOLT = 7/8" PER TABLE J3.4 — AISC 360-11 SPECIFICATION TABLE J3.4

FOR THE 3" HORIZ LEG, EDGE DISTANCE = 1.5" > 7/8" OK!

CHECK BEARING STR. FOR VERTICAL LEG AT 5/8" ATTACHMENT HOLE: — AISC 360-10 SPEC. J3.10



$\phi = 0.75$

$R_n = 1.2 l_c t F_u \leq 2.4 d t F_u$

$= 1.2(0.65625")(0.4375")(58ksi)$

$= 19.98^k$ ← GOVERNS

$\leq 2.4(5/8")(0.4375")(58ksi)$

$\leq 38.1^k$

$l_c = 1" - \frac{1}{2}\left(\frac{5}{8}" + \frac{1}{16}"\right) = 0.65625"$

$t = 0.4375"$ (FOR 7/16" ANGLE)

$F_u = 58ksi$ FOR A36 ANGLE

$d = 5/8"$

$\phi R_n = (0.75)(19.98^k)$

$= 14.99^k > 4.91^k$ OK!

TENSILE YIELDING OF CONNECTING ELEMENT — SPEC. J4.1 (J4-1)

$\phi = 0.90$

$R_n = F_y A_g$

A_g : GROSS AREA $= (3")(7/16") = 1.3125 in^2$

F_y : YIELD STR $= 36ksi$ (A36 ANGLE)

$\phi R_n = (0.90)(36ksi)(1.3125 in^2) = 42.53^k > P_u = 4.91^k$ OK!

TENSILE RUPTURE OF CONNECTING ELEMENT — (J4-2)

$\phi = 0.90$

$R_n = F_u A_e$

$A_e = A_n \leq 0.85 A_g$

$A_n = \left[3" - \left(\frac{5}{8}" + \frac{2}{16}"\right)\right](7/16") = 0.9844 in^2$

F_u : TENSILE STRENGTH $= 58ksi$ (A36 ANGLE)

$\phi R_n = (0.90)(58ksi)(0.9844 in^2) = 51.39^k > P_u = 4.91^k$ OK!

SEE HILTI CALCULATIONS FOR 1/2" EMBED PL STUD CHECK FOR PICK UP FORCE

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Specifier:	IBB	Project:	UNLV Rumble Strip
Address:		Sub-Project I Pos. No.:	
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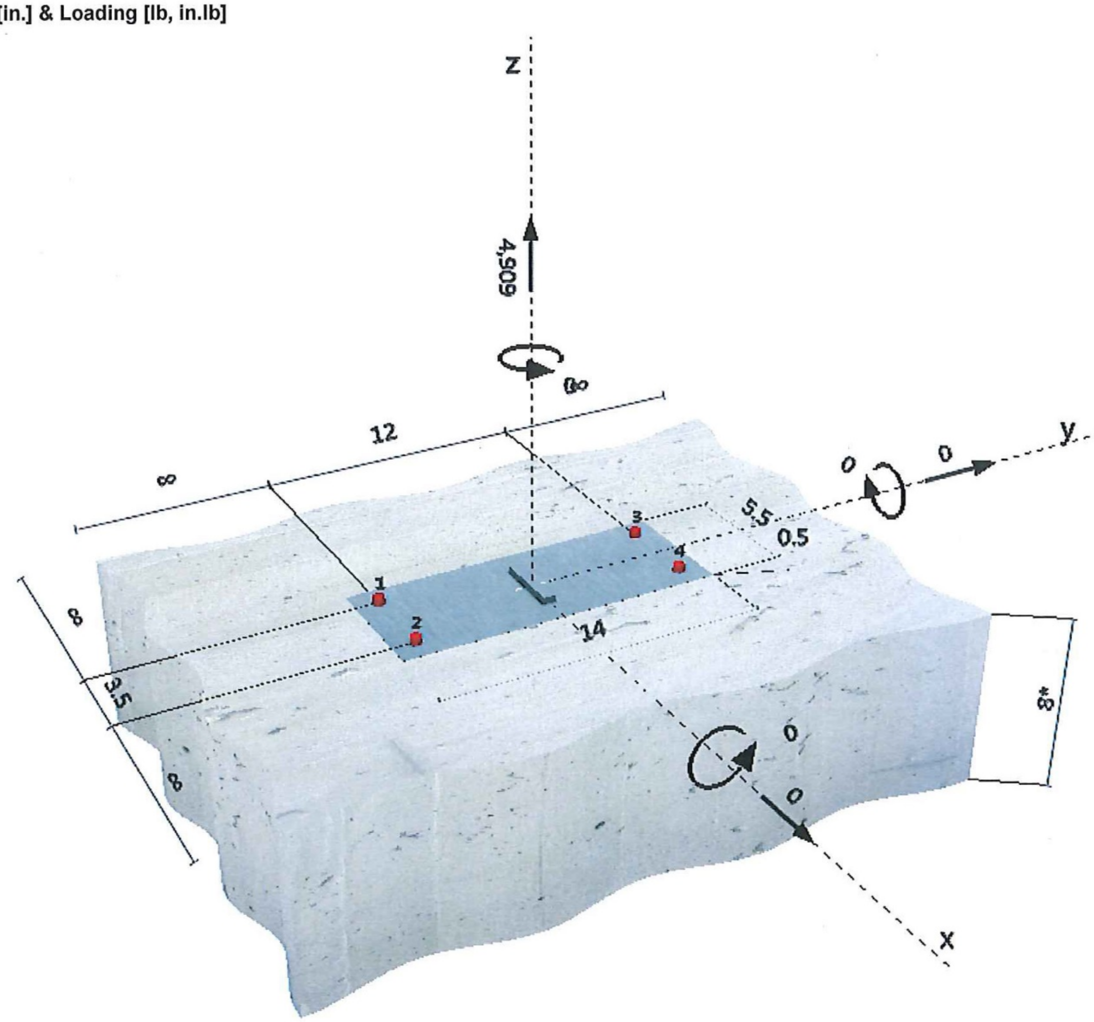
Specifier's comments:

1 Input data

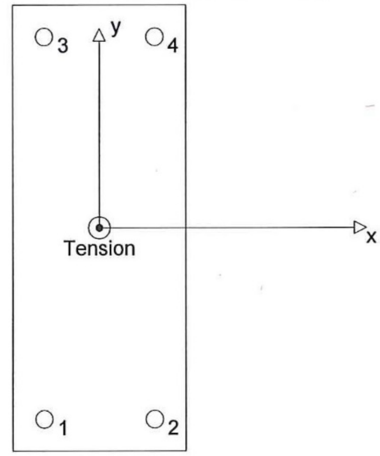
Anchor type and diameter:	**AWS D1.1 GR. B 1/2**
Effective embedment depth:	h_{ef} = 4.000 in.
Material:	
Proof:	Design method ACI 318-08 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 5.500 in. x 14.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	Rectangular plates and bars (AISC); (L x W x T) = 3.000 in. x 0.500 in. x 0.000 in.
Base material:	cracked concrete, , f_c' = 4500 psi; h = 8.000 in.
Reinforcement:	tension: condition B, shear: condition B; edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

Geometry [in.] & Loading [lb, in.lb]



2 Load case/Resulting anchor forces

Load case: Design loads



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1227	0	0	0
2	1227	0	0	0
3	1227	0	0	0
4	1227	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 4909 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	1227	9555	13	OK
Pullout Strength*	1227	14843	9	OK
Concrete Breakout Strength**	4909	23291	22	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-08 Eq. (D-3)
$\phi N_{sa} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.20	65000

Calculations

N_{sa} [lb]
12740

Results

N_{sa} [lb]	ϕ steel	ϕ N_{sa} [lb]	N_{ua} [lb]
12740	0.750	9555	1227

3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-08 Eq. (D-14)

$N_p = 8 A_{brg} f'_c$ ACI 318-08 Eq. (D-15)

$\phi\ N_{pN} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	f'_c [psi]
1.000	0.59	4500

Calculations

N_p [lb]
21204

Results

N_{pn} [lb]	$\phi_{concrete}$	$\phi\ N_{pn}$ [lb]	N_{ua} [lb]
21204	0.700	14843	1227

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-08 Eq. (D-5)

$\phi\ N_{cbg} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-08 Eq. (D-6)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-08 Eq. (D-9)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-08 Eq. (D-11)

$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-08 Eq. (D-13)

$N_b = k_c\ \lambda\ \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-08 Eq. (D-7)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
4.000	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ	f'_c [psi]
0.000	24	1	4500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
372.00	144.00	1.000	1.000	1.000	1.000	12880

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
33273	0.700	23291	4909

4 Shear load

	Load V_{ua} [lb]	Capacity ϕV_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

5 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!

Fastening meets the design criteria!

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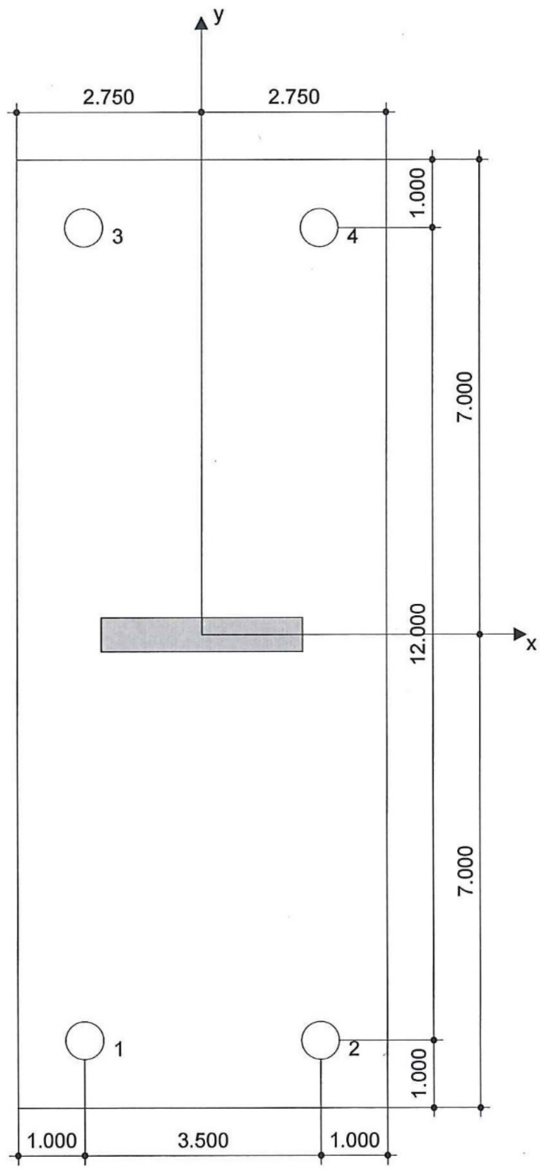
E-Mail:			

6 Installation data

Anchor plate, steel: -
Profile: Rectangular plates and bars (AISC); 3.000 x 0.500 x 0.000 in.
Hole diameter in the fixture: d_f = 0.563 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated
Drilling method: -
Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: AWS D1.1 GR. B 1/2
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 4.000 in.
Minimum thickness of the base material: 4.813 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-1.750	-6.000	-	-	-	-
2	1.750	-6.000	-	-	-	-
3	-1.750	6.000	-	-	-	-
4	1.750	6.000	-	-	-	-

7 Remarks; Your Cooperation Duties

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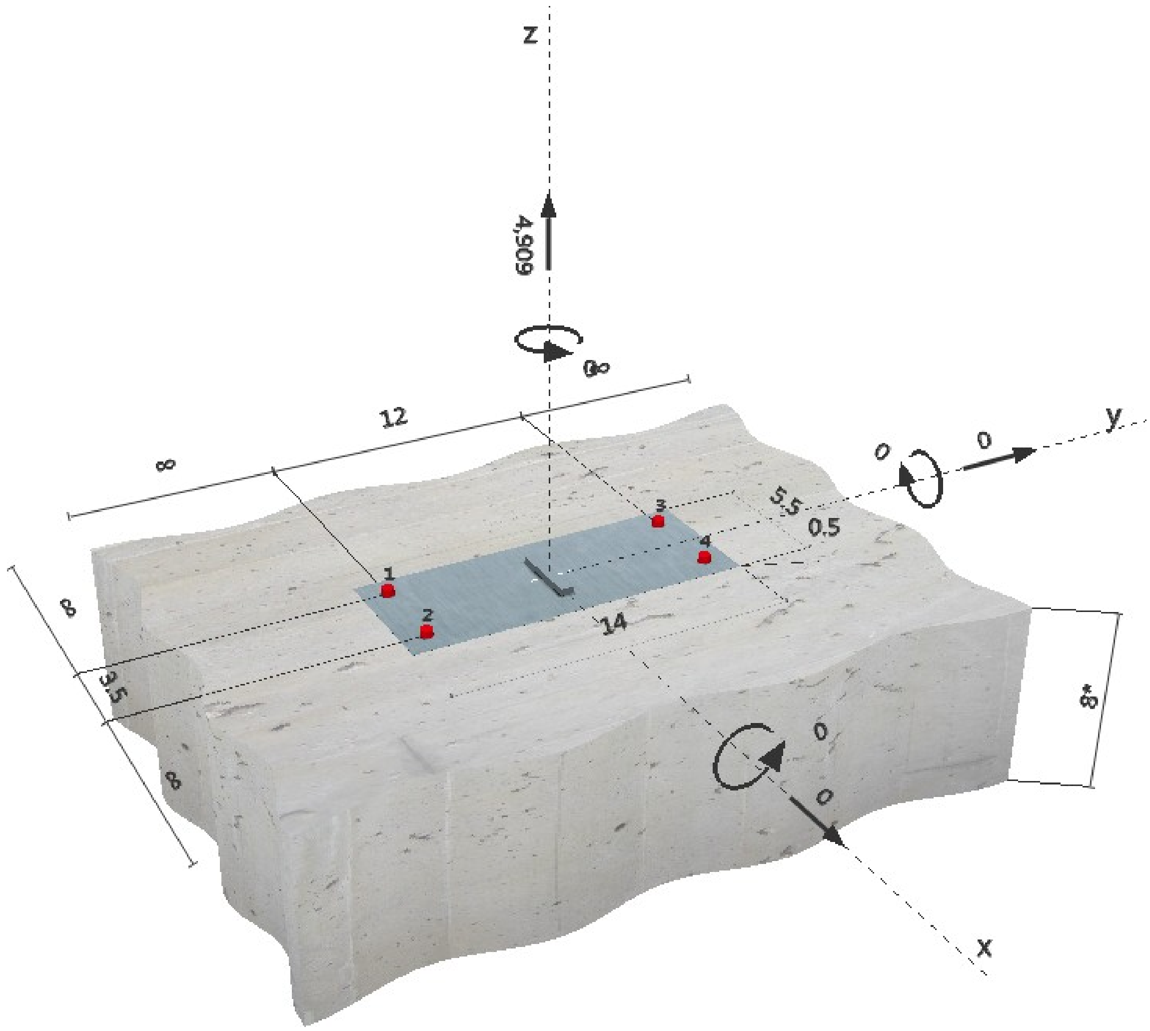
Phone I Fax:	|	Date:	2/12/2018
E-Mail:			

Specifier's comments:

1 Input data

Anchor type and diameter:	**AWS D1.1 GR. B 1/2**
Effective embedment depth:	h_{ef} = 4.000 in.
Material:	
Proof:	Design method ACI 318-08 / CIP
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.500 in.
Anchor plate:	l_x x l_y x t = 5.500 in. x 14.000 in. x 0.500 in.; (Recommended plate thickness: not calculated
Profile:	Rectangular plates and bars (AISC); (L x W x T) = 3.000 in. x 0.500 in. x 0.000 in.
Base material:	cracked concrete, , f_c' = 4500 psi; h = 8.000 in.
Reinforcement:	tension: condition B, shear: condition B;
	edge reinforcement: none or < No. 4 bar
Seismic loads (cat. C, D, E, or F)	no

Geometry [in.] & Loading [lb, in.lb]



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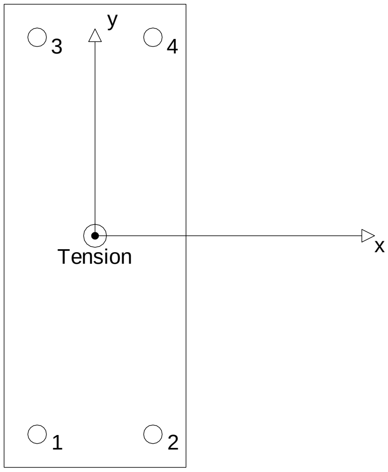
2 Load case/Resulting anchor forces

Load case: Design loads



Anchor reactions [lb]

Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	1227	0	0	0
2	1227	0	0	0
3	1227	0	0	0
4	1227	0	0	0

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 4909 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	1227	9555	13	OK
Pullout Strength*	1227	14843	9	OK
Concrete Breakout Strength**	4909	23291	22	OK
Concrete Side-Face Blowout, direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (anchors in tension)

3.1 Steel Strength

$N_{sa} = A_{se,N} f_{uta}$ ACI 318-08 Eq. (D-3)
$\phi N_{sa} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.20	65000

Calculations

N_{sa} [lb]
12740

Results

N_{sa} [lb]	ϕ_{steel}	ϕ N_{sa} [lb]	N_{ua} [lb]
12740	0.750	9555	1227

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3.2 Pullout Strength

$N_{pN} = \psi_{c,p} N_p$ ACI 318-08 Eq. (D-14)

$N_p = 8 A_{brg} f'_c$ ACI 318-08 Eq. (D-15)

$\phi N_{pN} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

Variables

$\psi_{c,p}$	A_{brg} [in.^{2}]	f'_c [psi]
1.000	0.59	4500

Calculations

N_p [lb]
21204

Results

N_{pn} [lb]	$\phi_{concrete}$	ϕN_{pn} [lb]	N_{ua} [lb]
21204	0.700	14843	1227

3.3 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b$ ACI 318-08 Eq. (D-5)

$\phi N_{cbg} \geq N_{ua}$ ACI 318-08 Eq. (D-1)

A_{Nc} see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-08 Eq. (D-6)

$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-08 Eq. (D-9)

$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-08 Eq. (D-11)

$\psi_{cp,N} = \mathrm{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-08 Eq. (D-13)

$N_b = k_c \lambda \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-08 Eq. (D-7)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
4.000	0.000	0.000	∞	1.000

c_{ac} [in.]	k_c	λ	f'_c [psi]
0.000	24	1	4500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
372.00	144.00	1.000	1.000	1.000	1.000	12880

Results

N_{cbg} [lb]	$\phi_{concrete}$	ϕN_{cbg} [lb]	N_{ua} [lb]
33273	0.700	23291	4909

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4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	N/A	N/A	N/A	N/A
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength*	N/A	N/A	N/A	N/A
Concrete edge failure in direction **	N/A	N/A	N/A	N/A

* anchor having the highest loading **anchor group (relevant anchors)

5 Warnings

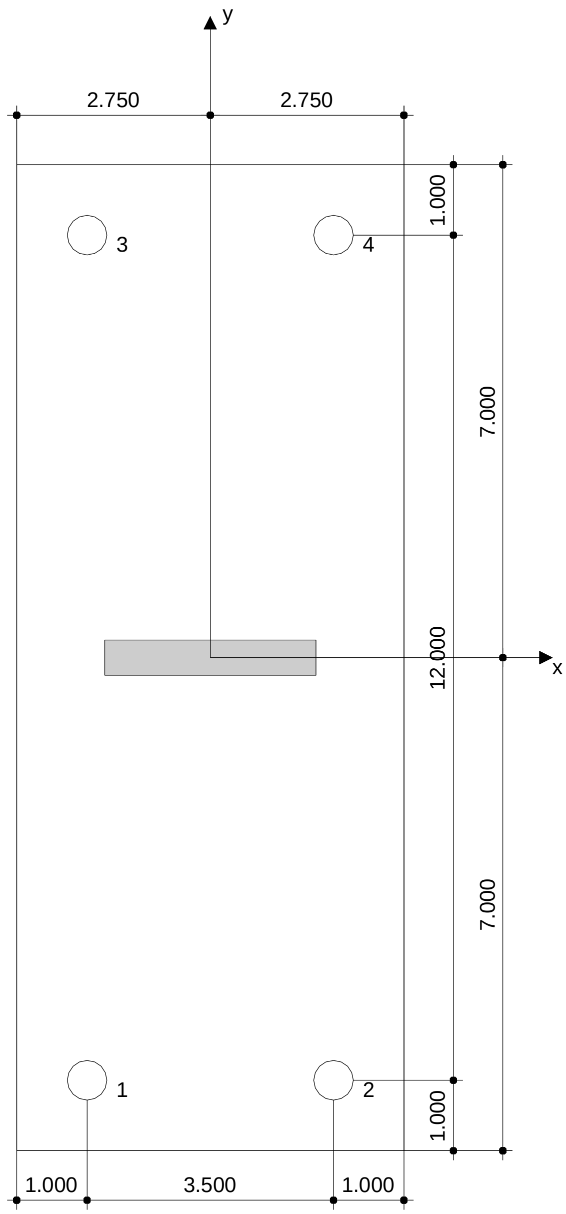
- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!

Fastening meets the design criteria!

6 Installation data

Anchor plate, steel: -
Profile: Rectangular plates and bars (AISC); 3.000 x 0.500 x 0.000 in.
Hole diameter in the fixture: d_f = 0.563 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated
Drilling method: -
Cleaning: No cleaning of the drilled hole is required

Anchor type and diameter: AWS D1.1 GR. B 1/2
Installation torque: -
Hole diameter in the base material: - in.
Hole depth in the base material: 4.000 in.
Minimum thickness of the base material: 4.813 in.



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-1.750	-6.000	-	-	-	-
2	1.750	-6.000	-	-	-	-
3	-1.750	6.000	-	-	-	-
4	1.750	6.000	-	-	-	-

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APPENDIX B
Demand-Responsive Rumble Strip (DRRS) Structural Analysis Overview for the Third Design

Demand-Responsive Rumble Strip (DRRS) Structural Analysis Overview

The proposed design is composed of five rumble strips, Figure 1. The rumble strips will be typically at roadway level. If there is a need to slow passing vehicles down, the rumble strips will be lowered to a pre-set level.

Each rumble strip is activated by three hydraulic actuators, Figure 2. These hydraulic actuators are single-acting units. While pressure moves, and maintains, the rumble street up to the roadway level, an internal spring, placed inside each hydraulic actuator, pushes it down to the desired depth (0.5" below the roadway).

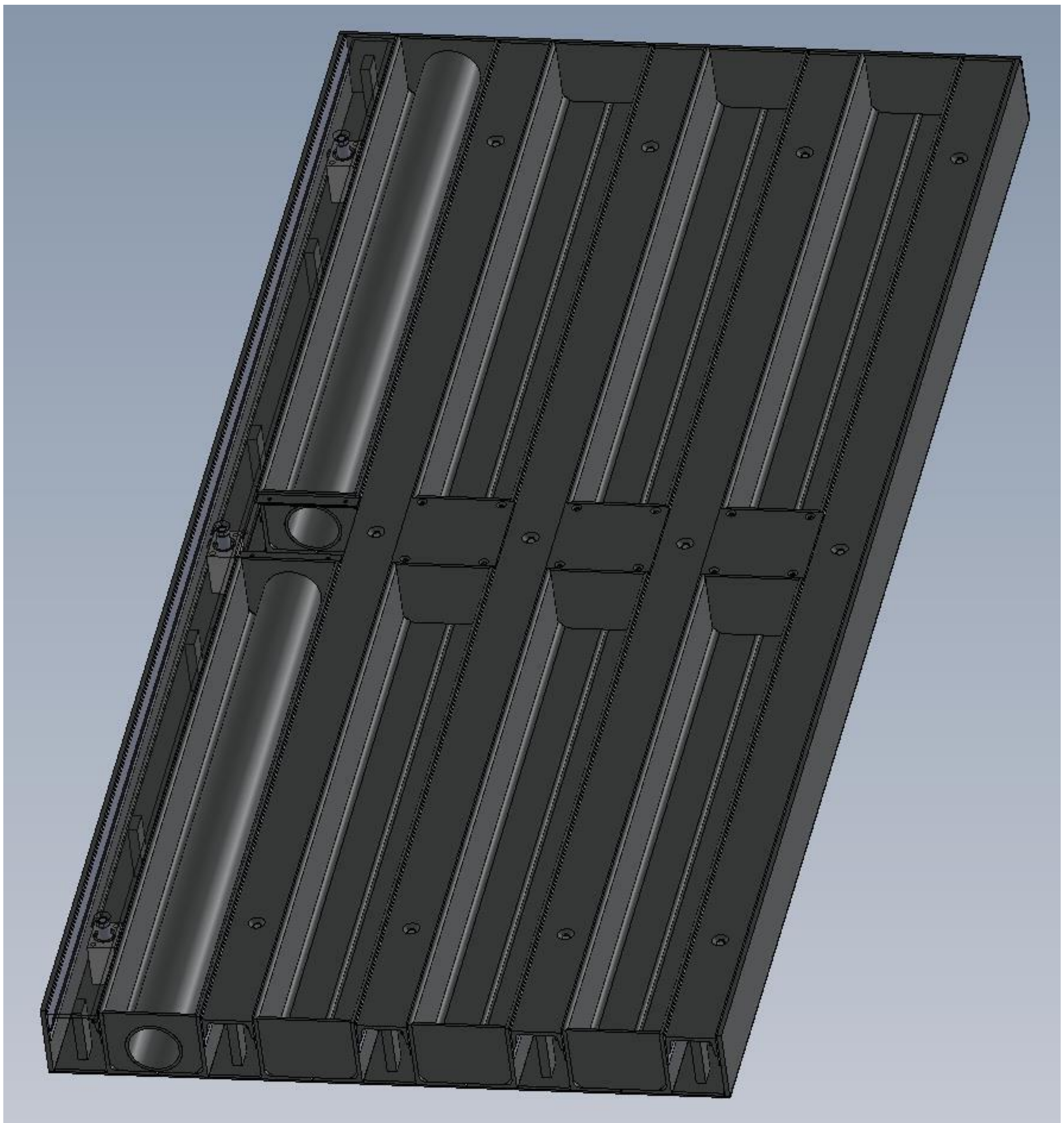


Figure 1. Demand-Responsive Rumble Strip System (End of the Box Is Hidden and the Top Channel of the Left-most Rumble Strip Unit Is Rendered Transparent to Ease Viewing)

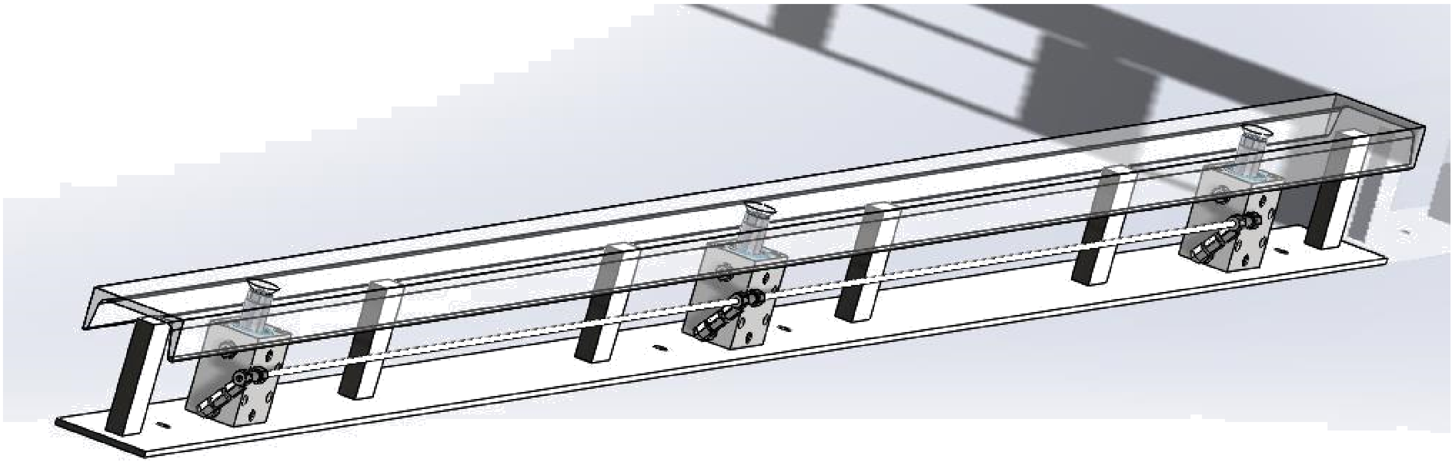


Figure 2 (a). Rumble Strip. The Strip is activated using Three Hydraulic Cylinders. It Is Supported by Six Columns when It Is at the Lowest Position

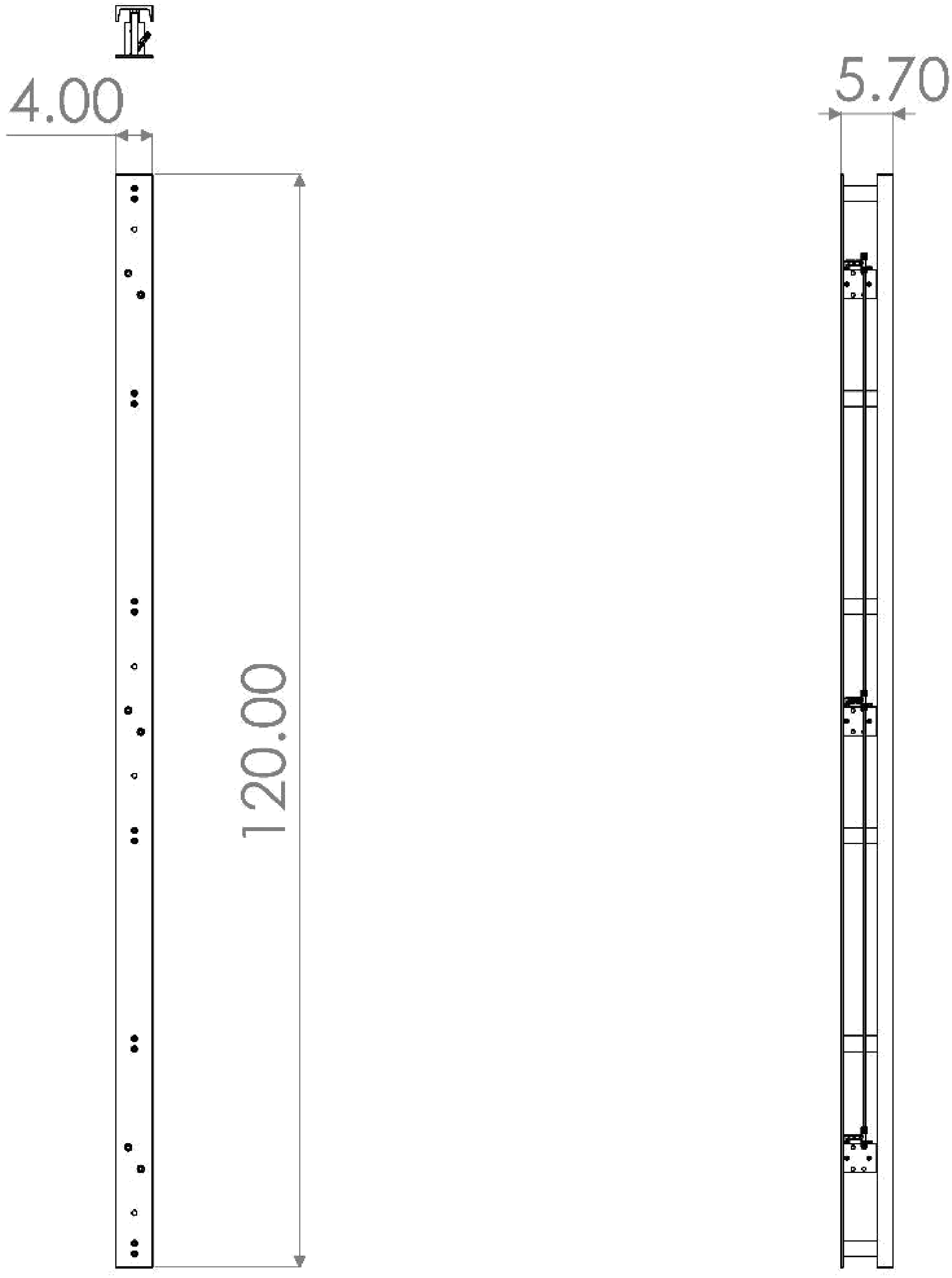


Figure 2 (b). Side and Top Views of a Rumble Strip.

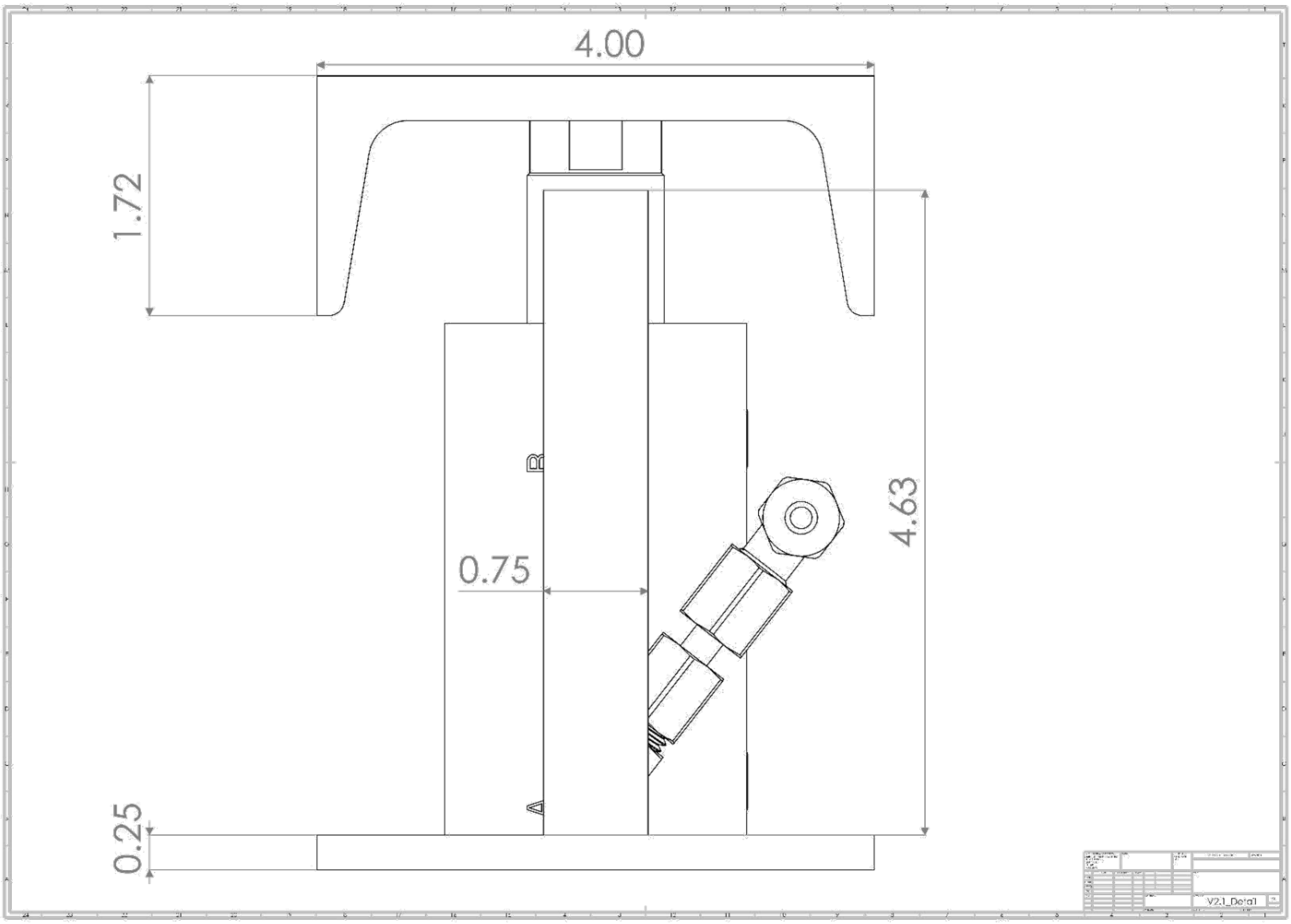


Figure 2 (c). Front View of the Rumble Strip.

The rumble strip units are placed within a steel box, Figure 3, which is attached to the roadway using studs and epoxy. The rumble strips are separated by spacer blocks. The two spacer blocks on the left side of Figure 3 have 4.5 inch pipe to allow routing of the hydraulic units hoses. Each block is manufactured from steel sections in the shape of open trough to allow pouring concrete into it. Concrete will add stability to the system and reduce the possibility of slippage

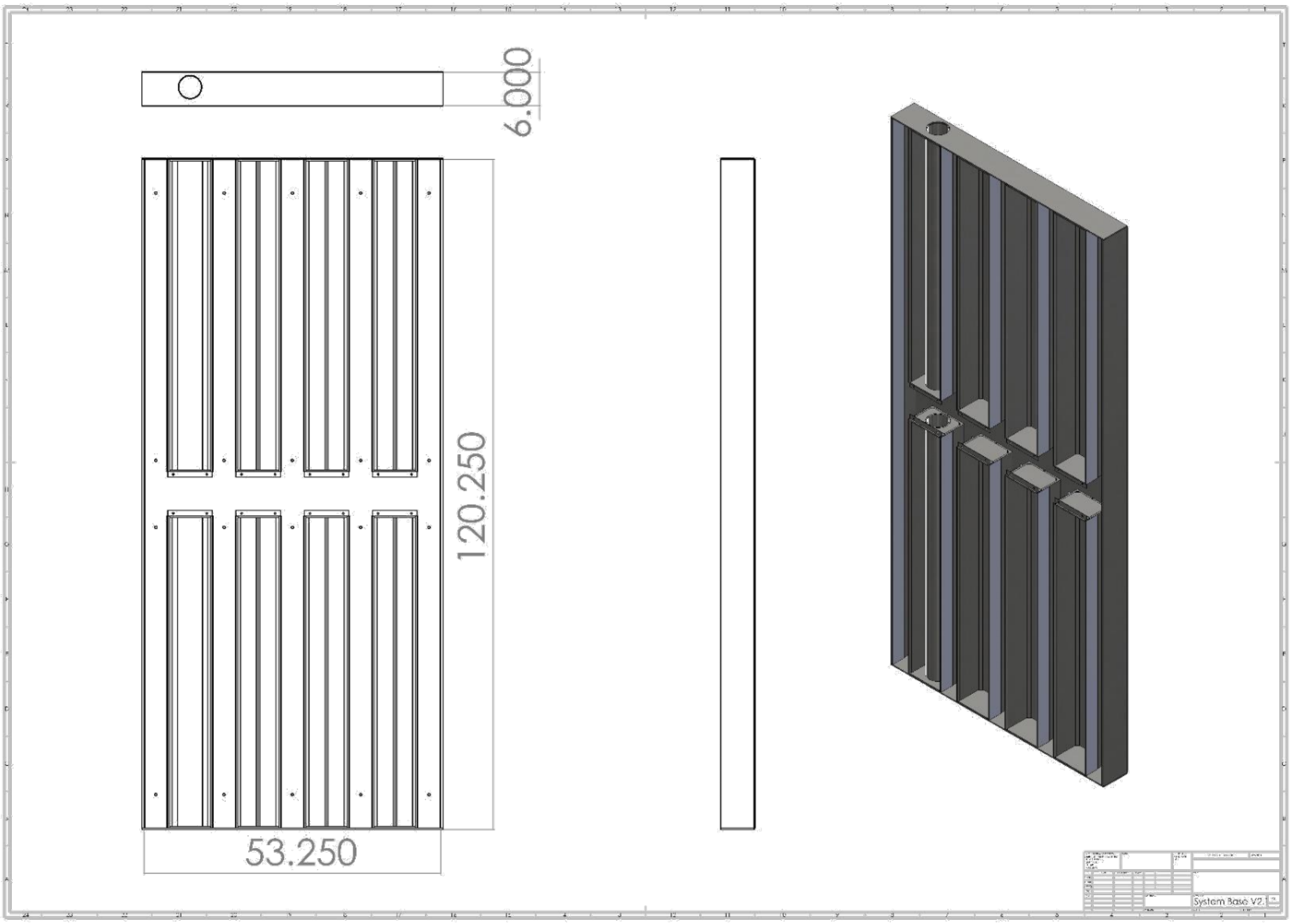


Figure 3. Demand-Responsive Rumble Strip System Box

Structural Analysis

The following is a structural analysis of the DRRS system under the load induced by a fully-loaded truck that is braking over a rumble strip system. For simplicity, a rumble strip with the surrounding separating blocks are considered, Figure 4. The cavities of the support blocks are filled with concrete.

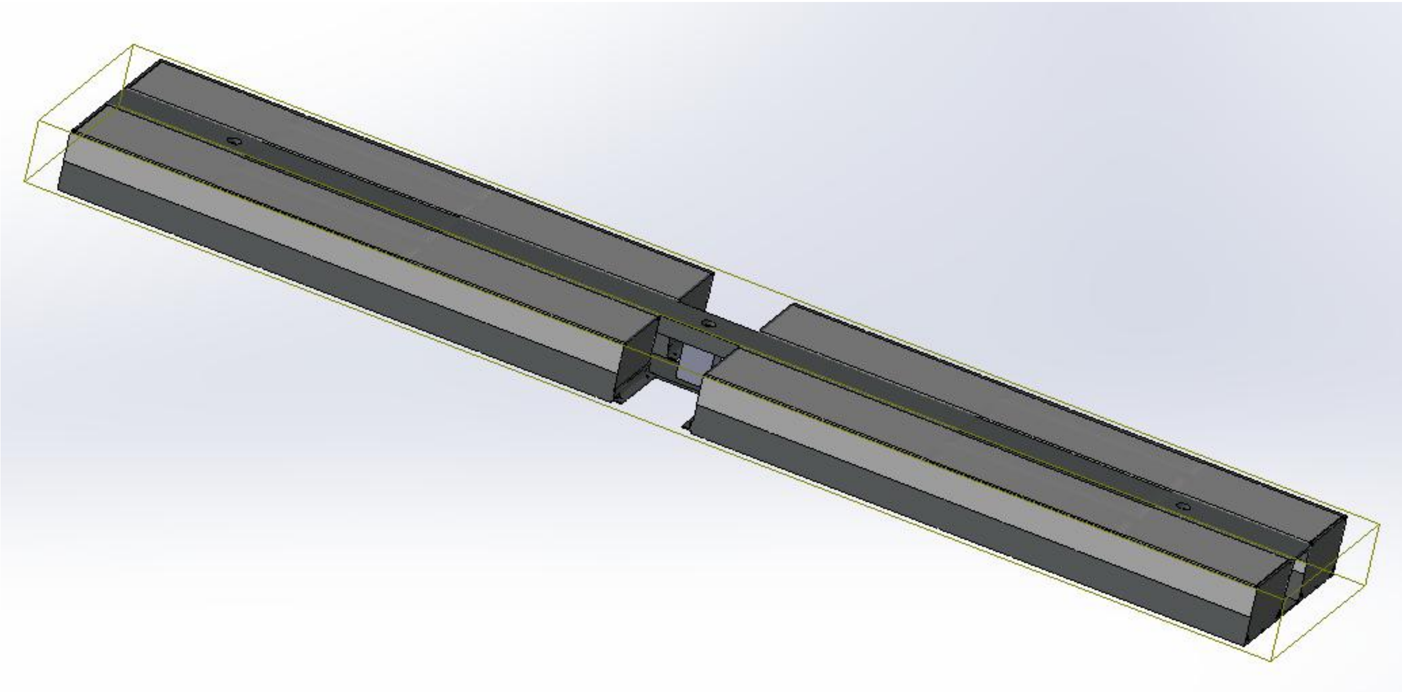


Figure 4. Demand-Responsive Rumble Strip with Four Spacer Blocks

Truck Loading

The weights and spacing of axles and wheels for the design truck shall be as specified in Figure 5. A dynamic load allowance shall be considered as Dynamic load allowance (IM). The spacing between the two 32.0 kip axles shall be varied between 14.0 ft and 30.0 ft. The transverse spacing of tires is as 6.0 ft.

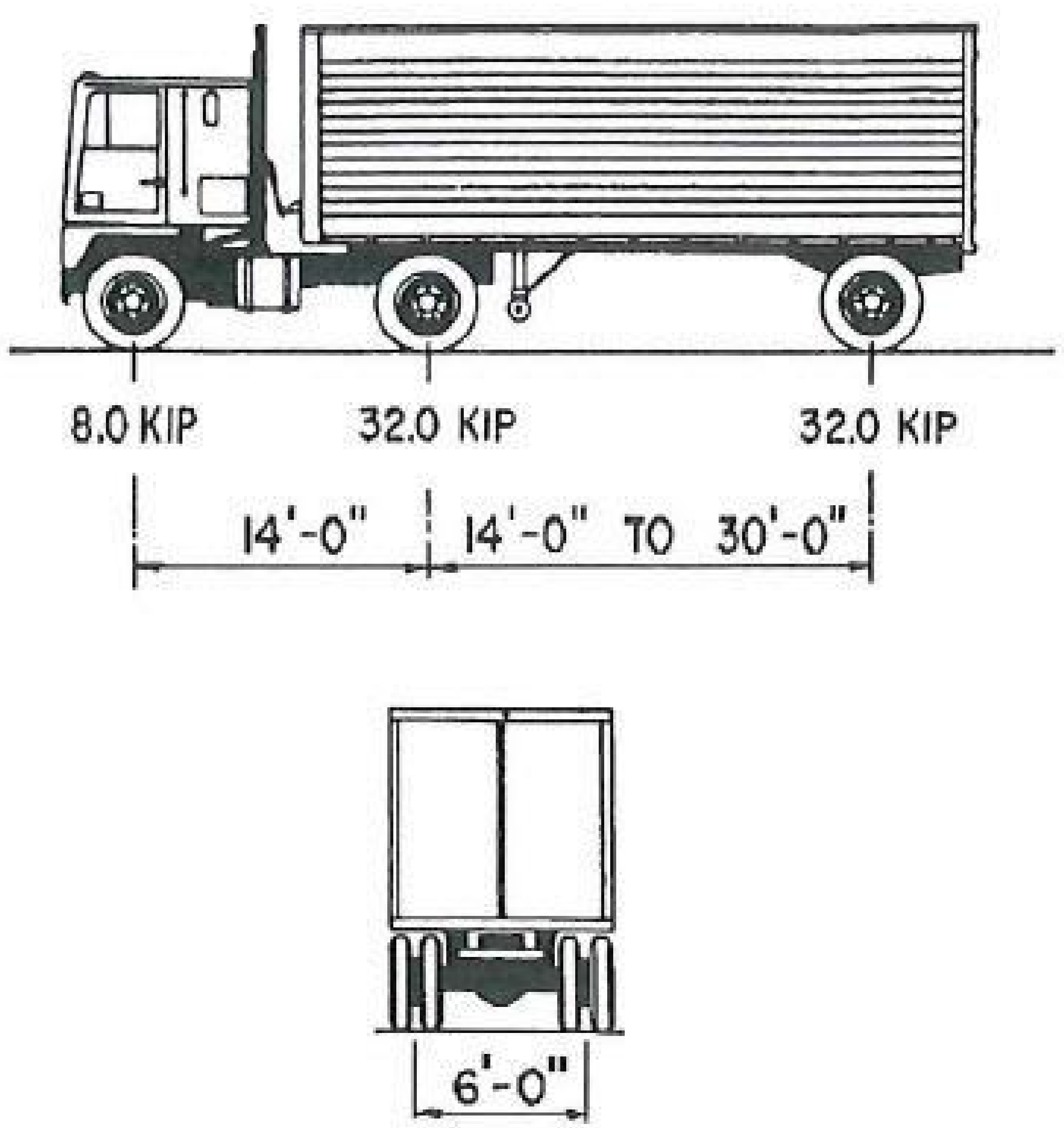


Figure 5. Characteristics of the Loading Truck [20]

Tire Contact Area

The tire contact area of a wheel consisting of one or two tires shall be assumed to be a 20” x 10“ rectangle. The twenty-inch side of the contact area is parallel to the axle. It is also assumed that:

- The tire pressure is assumed to be uniformly distributed over the continuous contact area.
    - On interrupted surfaces, the tire pressure is uniformly distributed over the actual contact area within the footprint with the pressure increased in the ratio of the specified to actual contact areas [20].

Case Study #1: Load is shared by the Rumble Strip Unit and the Support Blocks

When the strips are at the roadway level, the most severe loading case may happen when a truck, as described above, brakes over the strip. In this case, the strip and the neighboring spacer blocks will carry the load of the axles in addition to braking load. Tire contact area is shown in Figure 6. This scenario indicates that the tire pressure is divided between the rumble strip unit and the two supporting blocks.

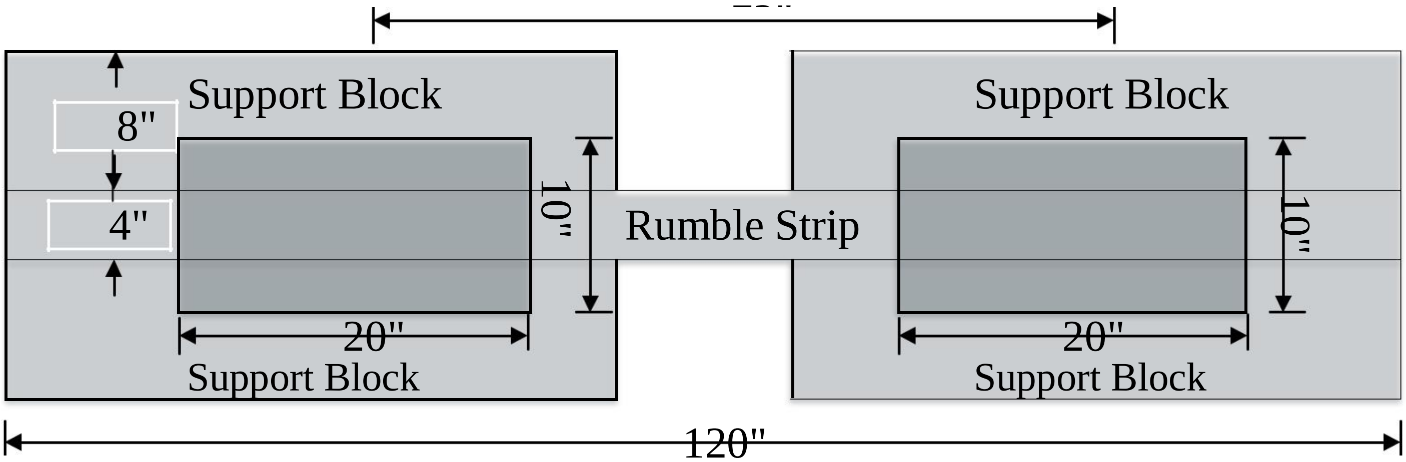


Figure 6. Tire Contact Area of the Case Study

In th~~is case,~~ the normal pressure each tire is applying is equal to:

$= 2(10)(20)^{32000} = 80$

A load factor of 1.33 is used, increasing the normal pressure to, 106.4 psi

A second load is generated in the tangential direction (parallel to the surface of the roadway) due to braking. According to Section 3.6.4 of AASHTO 2012 LRFD Bridge Design Specifications [20], the braking force shall be assumed as 25 percent of the axle weights of the design truck Therefore, tangential pressure due to the stopping of the truck is 100 psi.

Braking load is verified by calculating the force due to decelerating the vehicle

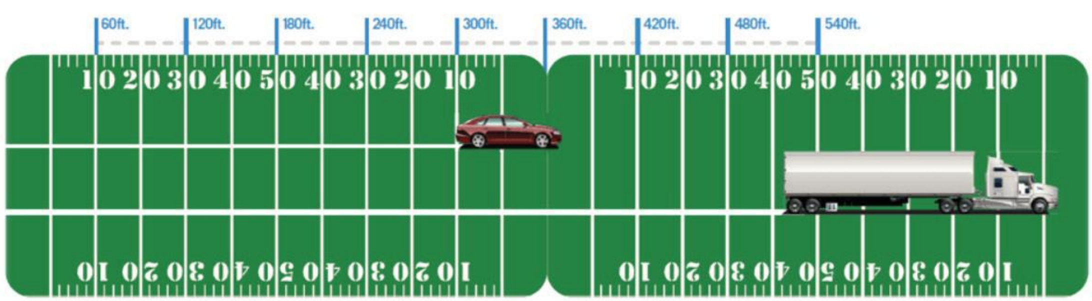


Figure 7. Comparison of Stopping Distances at 65 mph [21]

Figure 7 shows that stopping distance for a truck is, $\overline{d\ 525}\ ft$. Assuming that the deceleration is constant, it can be calculated for the case when the velocity of the truck before the start of the deceleration is, $V_0\ \overline{65}\ mph$. Using principles of kinematics,

$$T = -\frac{V_0}{a}$$

$$\Delta d = \frac{aT^2}{2} + V_0 T$$

where T is the stopping time

a is the deceleration

Merging the two equations,

$$\Delta d = \frac{aT^2}{2} + V_0 T = \frac{a\left(-\frac{V_0}{a}\right)^2}{2} + V_0\left(-\frac{V_0}{a}\right) = -\frac{V_0^{\ 2}}{2a}$$

Rearranging the above equation,

$$a = -\frac{V_0^{\ 2}}{2\Delta d} = -\frac{(65\ mph)^2}{2*525\ ft} = -8.65\frac{ft}{s^2}$$

The associated force is,

$$F = ma = 8000\ lb\ \frac{8.65 ft/s^2}{32.17\ ft/s^2} = 21508\ lb$$

These results are consistent with [20].

Finite Element Analysis of the Rumble Strip System

For analysis purpose, only the middle rumble strip and its surrounding spacer blocks are placed inside the box, Figure
8. This can be considered as a conservative approach as the other spacer blocks and rumble strips will add stability to the system.

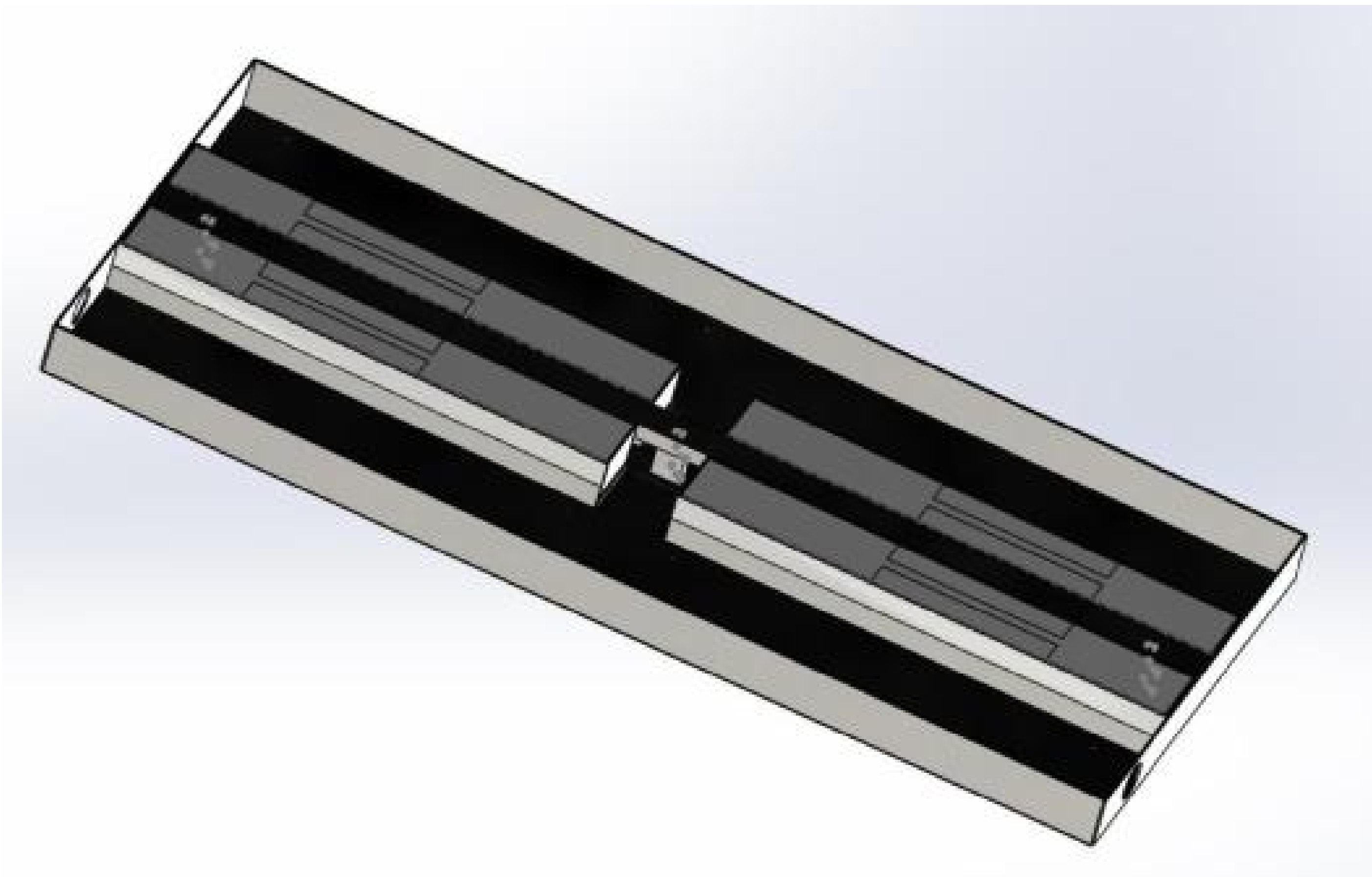


Figure 8. Demand-Responsive Rumble Strip with Four Spacer Blocks

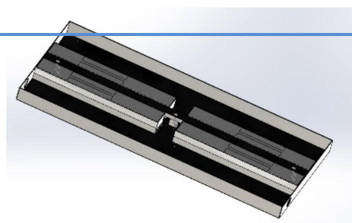
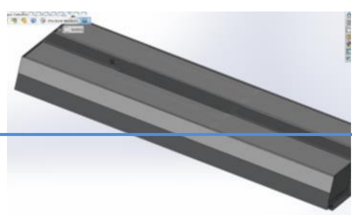
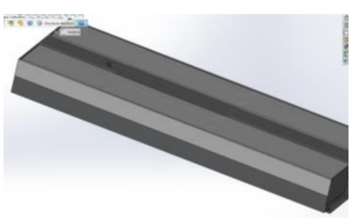
Study Properties

Study name	Truck Loading plus Braking
Analysis type	Static
Mesh type	Solid Mesh
Thermal Effect:	On
Thermal option	Include temperature loads
Zero strain temperature	298 Kelvin
Include fluid pressure effects from SOLIDWORKS Flow Simulation	Off
Solver type	Direct sparse solver
Inplane Effect:	Off
Soft Spring:	Off
Inertial Relief:	Off
Incompatible bonding options	Automatic
Large displacement	Off
Compute free body forces	On
Friction	Off
Use Adaptive Method:	Off
Result folder	SOLIDWORKS document (H:\My Drive\Rumble Strips\Design 2 (1)\rumble strip v2\V2.1 FEA)

Units

Unit system:	English (IPS)
Length/Displacement	in
Temperature	Fahrenheit
Angular velocity	Hertz
Pressure/Stress	psi

Material Properties

Model Reference	Properties	Components
	Name: Plain Carbon Steel Linear Elastic Model type: Isotropic Default failure criterion: Max von Mises Stress Yield strength: 31994.5 psi Tensile strength: 57989.9 psi Elastic modulus: 3.04579e+007 psi Poisson's ratio: 0.28 Mass density: 0.281793 lb/in^3 Shear modulus: 1.1458e+007 psi Thermal expansion coefficient: 7.22222e-006 /Fahrenheit	All components are assumed to be made of Plain Carbon Steel unless specified otherwise
 	Name: PC Linear Elastic Model type: Isotropic Default failure criterion: Max von Mises Stress Tensile strength: 5801.51 psi Elastic modulus: 349541 psi Poisson's ratio: 0.3897 Mass density: 0.0386562 lb/in^3 Shear modulus: 125052 psi	Plastic strip spacers are attached to the space block to reduce friction during the motion of the rumble strip
	Name: Concrete 1 [22] Linear Elastic Model type: Isotropic Default failure criterion: Tensile strength: 500 psi Compressive strength: 6000 psi Elastic modulus: 4e+006 psi Poisson's ratio: 0.22 Mass density: 0.055 lb/in^3 Shear modulus: 2.4e+006 psi Thermal expansion coefficient: 5e-006 /Fahrenheit	As mentioned earlier, concrete filling is used inside the support blocks

Boundary Conditions

The bottom and the side of the box in addition to the sides of the spacer blocks are fully restricted to simulate the support the blocks receive from the roadway and the box respectively, Figure 9.

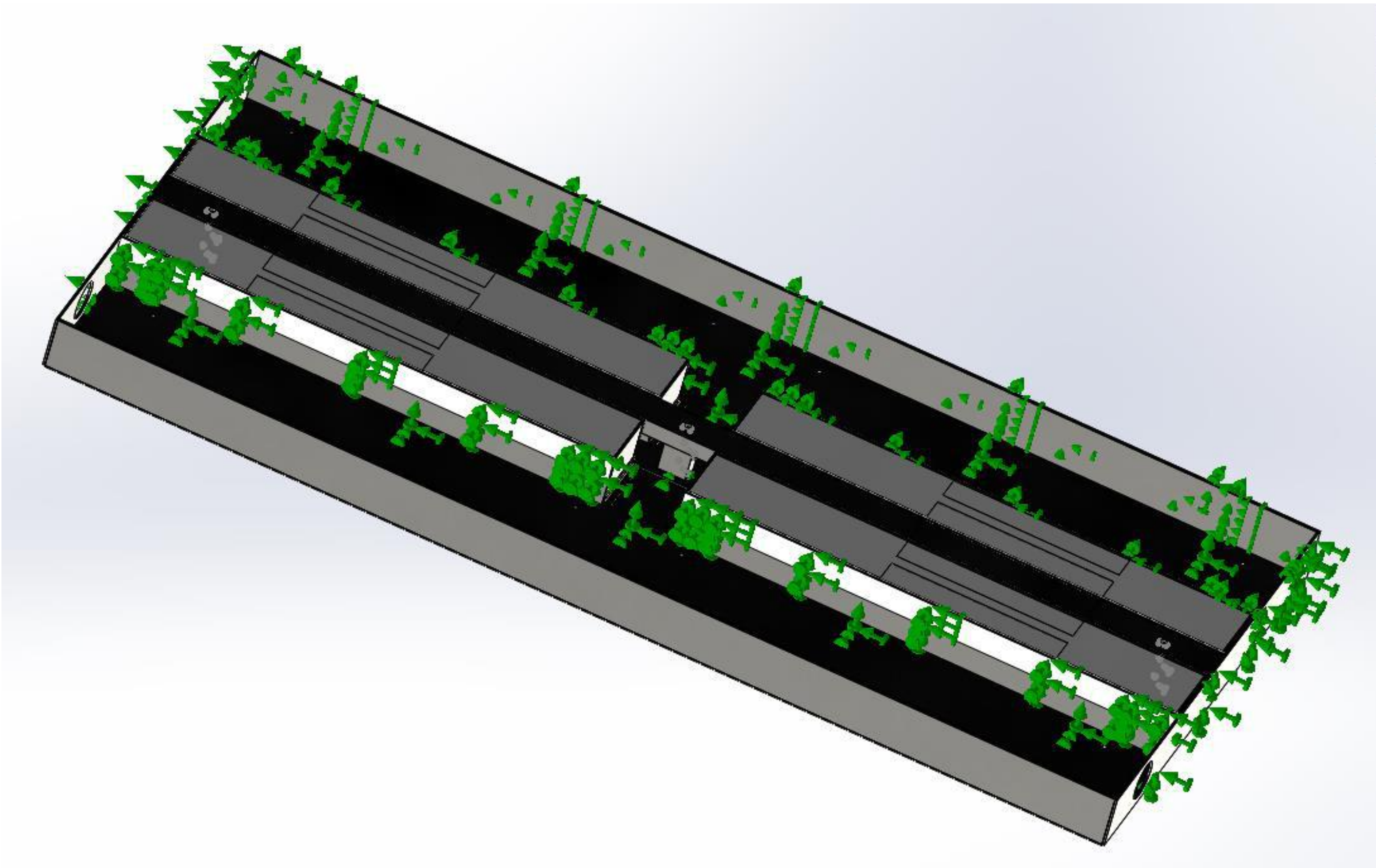


Figure 9. Boundary Conditions of the Case Study

Applied Loads

Loads are applied to the upper surface of the rumble strips and support blocks as specified above and shown in Figure 9.

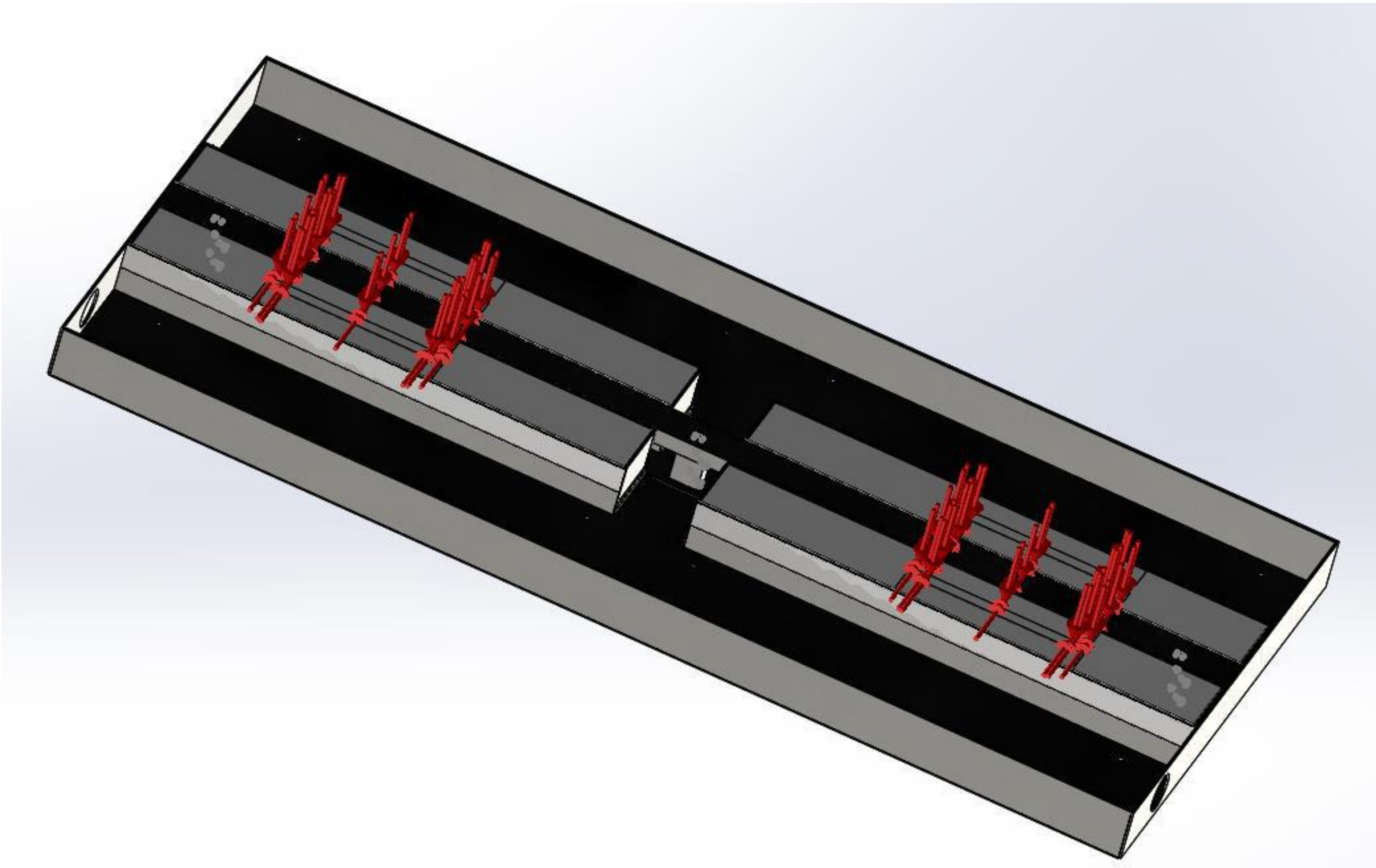


Figure 10. Applied loads in the Normal (Weight of the Truck) and Tangential (Braking) Directions

Contact Information

All parts that will be welded or bolted are bonded in the model using compatible meshes.

Mesh

Generating the mesh was challenging as the thickness The table below lists the mesh information. Figure 11 shows the overall mesh while Figures 12 and 13 show detailed views of the mesh used to describe the rumble strip top channel, actuator, spacer blocks, and the box bottom.

Mesh type	Solid Mesh
Mesher Used:	Standard mesh
Automatic Transition:	Off
Include Mesh Auto Loops:	Off
Jacobian points	4 Points
Element Size	1.49125 in
Tolerance	0.0745625 in
Mesh Quality Plot	High
Total Number of Elements	194765

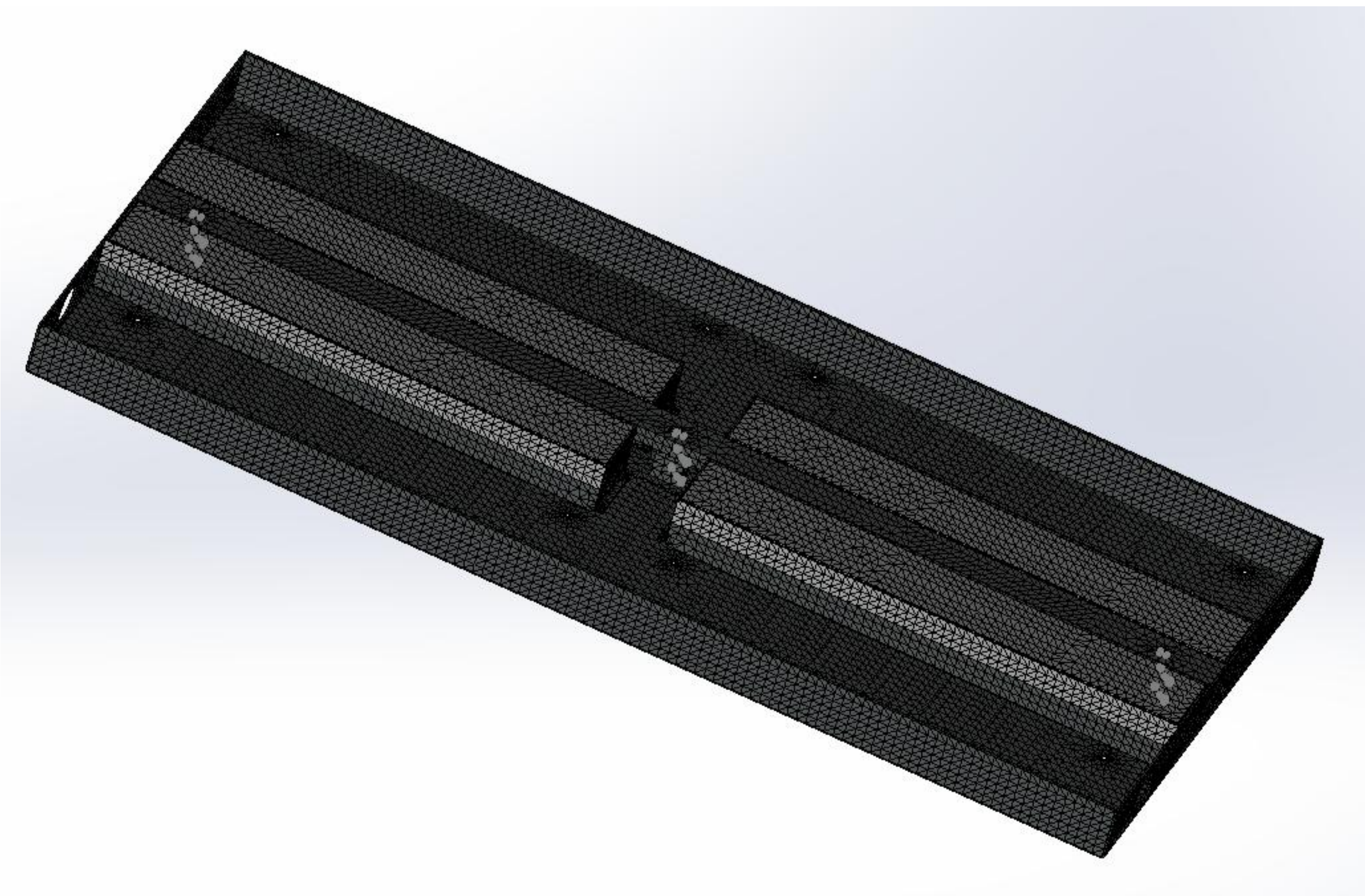


Figure 11. Finite Element Mesh Used to Describe the Problem

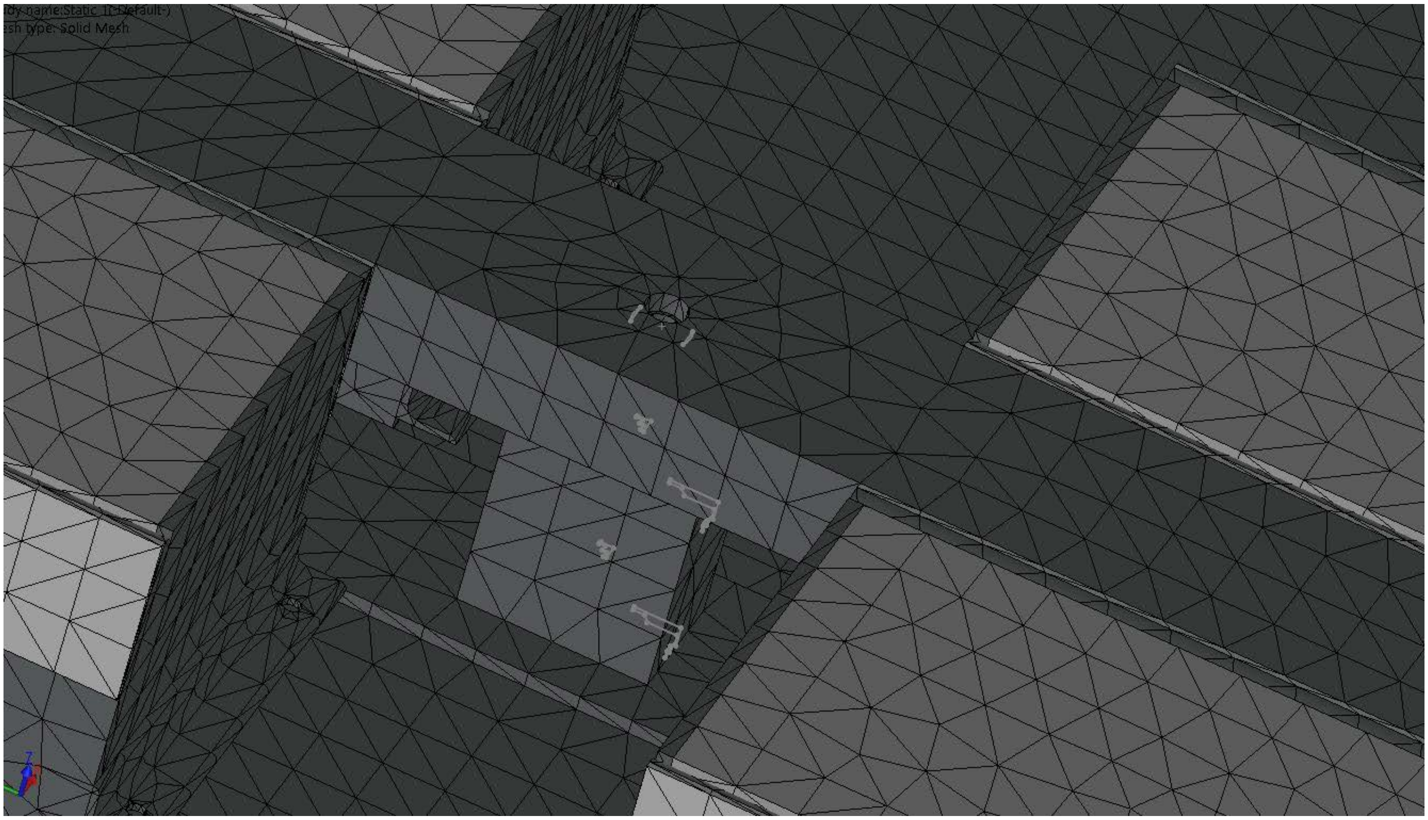


Figure 12. Mesh Detail near the Center of the Rumble Strip

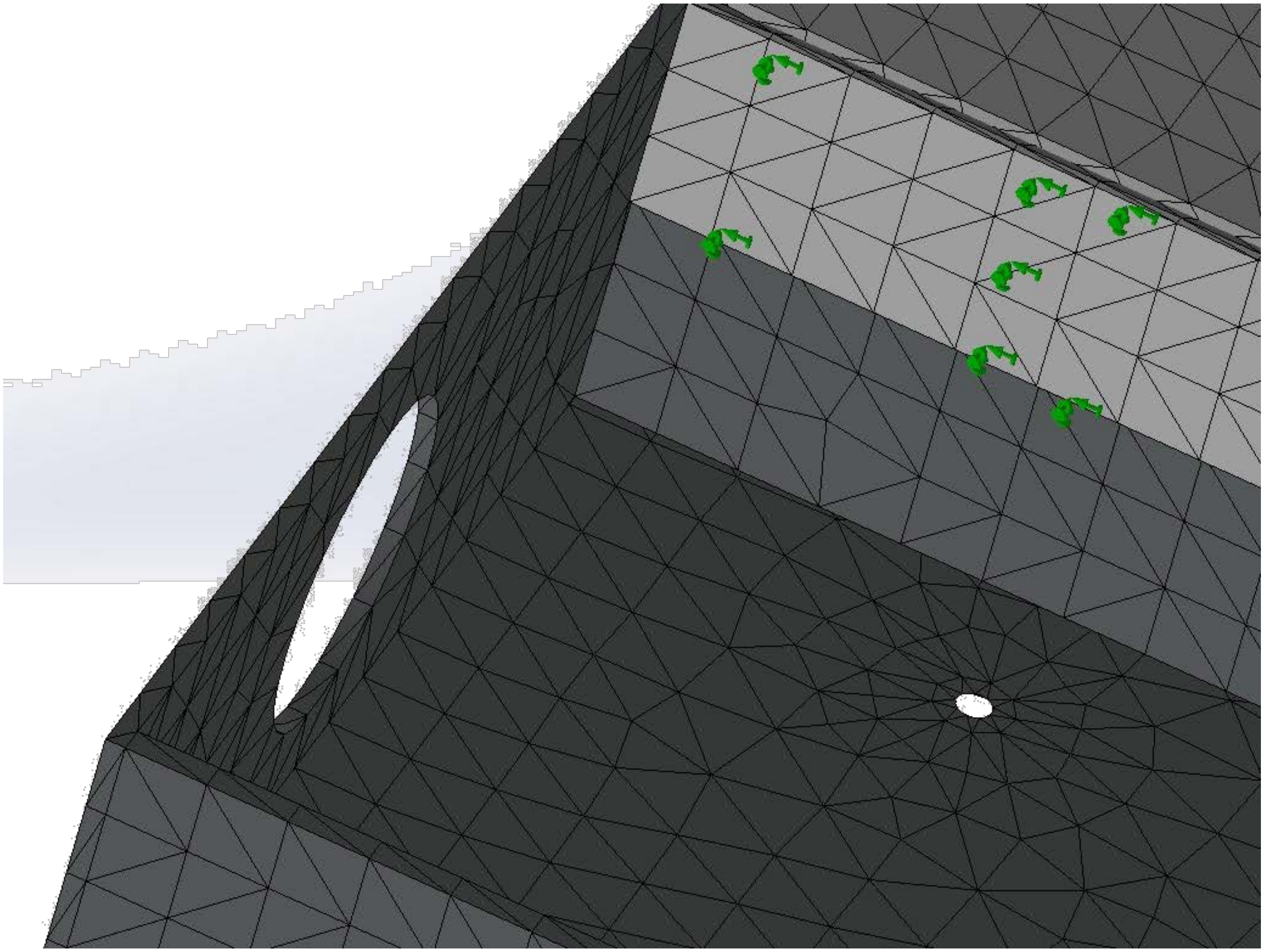
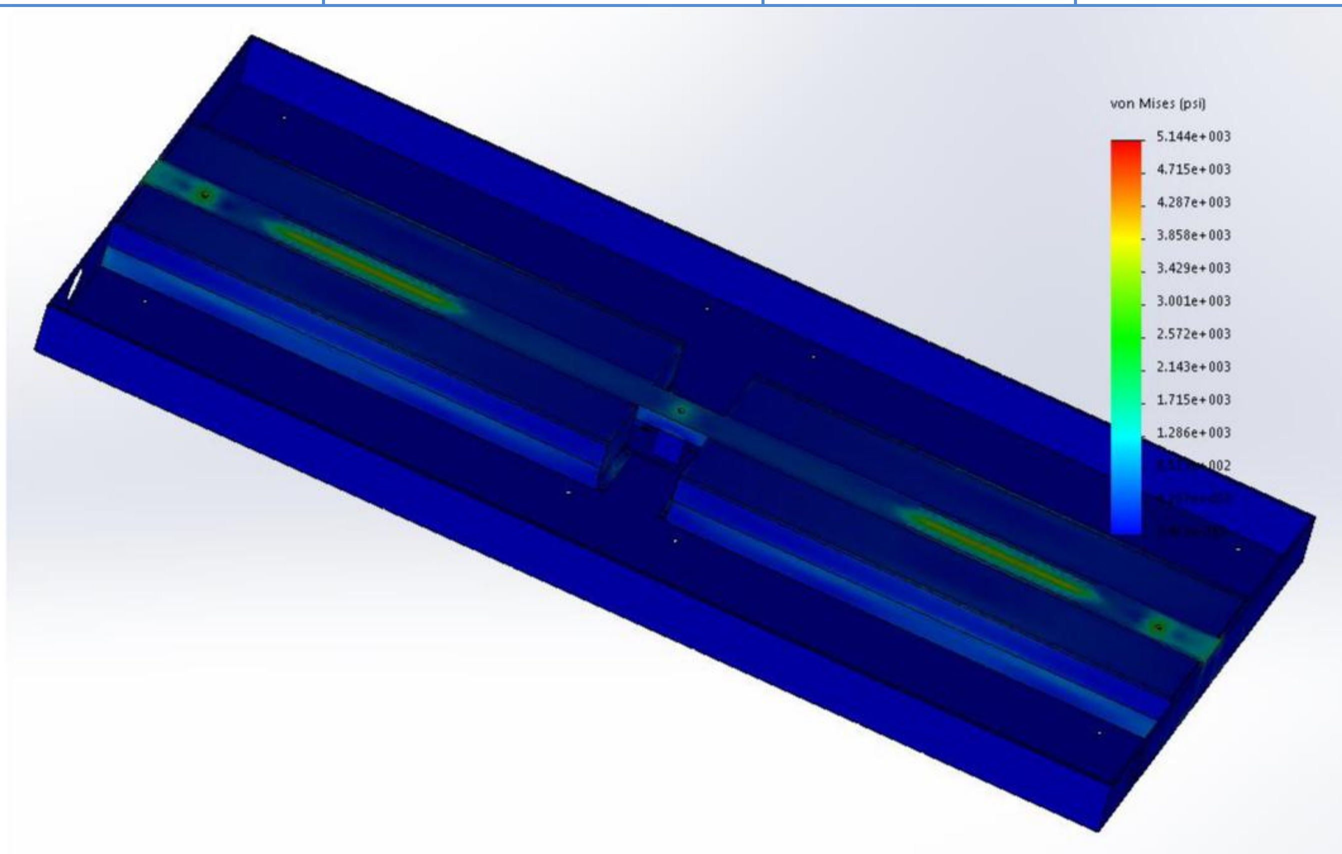


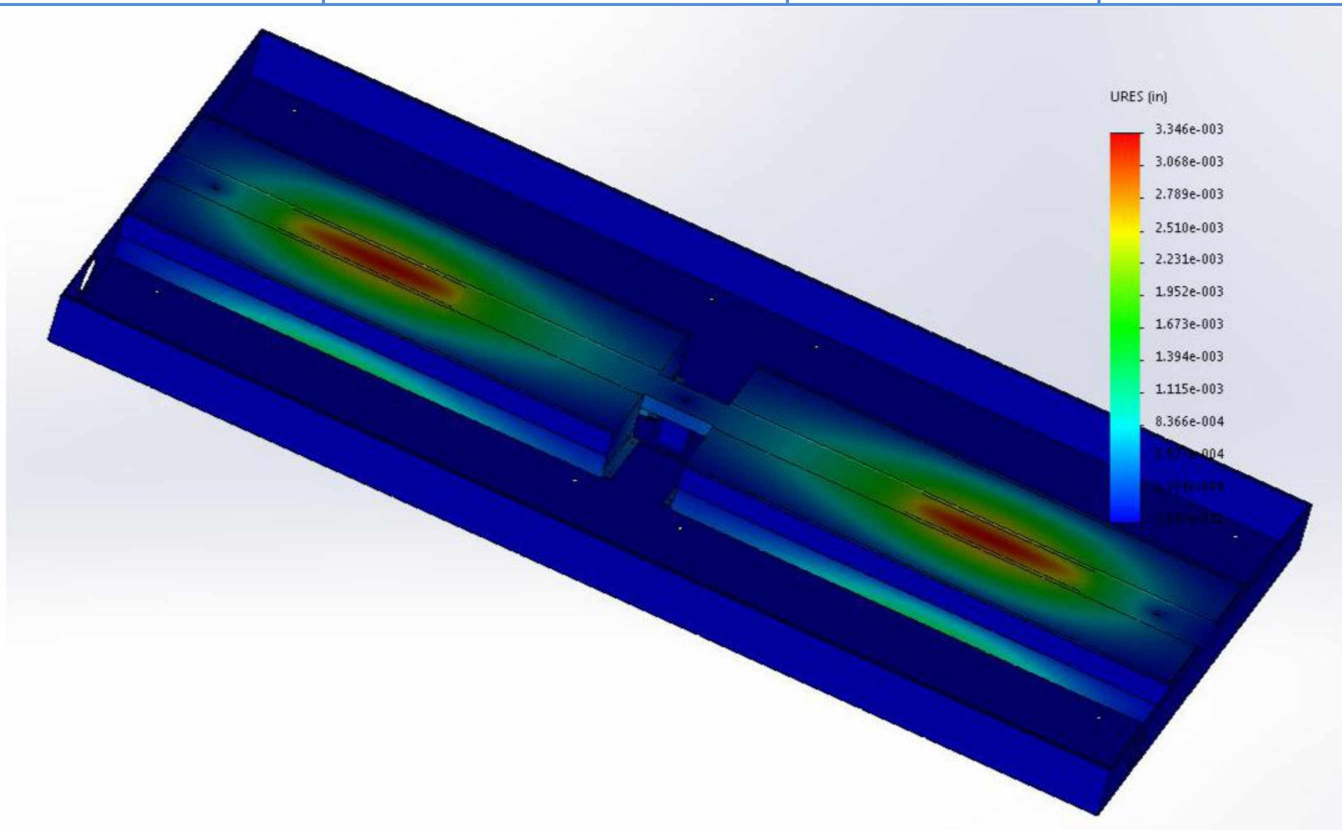
Figure 13. Mesh Detail near the end of the Rumble Strip

Study Results

Name	Type	Min	Max
Stress1	VON: von Mises Stress	0.000e+000psi	5.144e+003psi



Name	Type	Min	Max
Displacement1	URES: Resultant Displacement	0.000e+000in	3.346e-003in



Conclusions

As the results show, the loads induce minimum deformation that occurs in the unsupported portion of the rumble strip upper channel.

The maximum von Mises stress recorded is 3784 psi in the rumble strips. Concrete experiences compressive stresses in the order of 300 psi.

It is reasonable to assume that the failure will be due to fatigue loading as the rumble strip will be subjected to repeated loads. The approach suggested by [23] is followed in this section. The process is started by calculating the endurance limit of the steel using the following formula,
(0.5)(58000)(1)(0.8)(0.77)(1)(0.753) = 13450

where,

Su Ultimate strength

CL Loading factor

CG Gradient factor

Cs Surface factor

CT Temperature factor

CR Reliability factor, which is chosen as 99.9%

In this case study, the rumble strips will experience load that varies from zero to maximum. Therefore, the amplitude and mean of stresses will be equal to each other or,

$$\sigma_m = \sigma_a = 5144/2 = 2572\ psi$$

Using Goodman line, the factor of safety can be calculated as:
$f_s = \{(S_n)(S_u)/(S_o + S_u)\}/\sigma_a = 10918/2572 = 4.25$
This factor of safety may be adequate to meet other unexpected loads the system may experience. The highest stress is below the 15% *Dynamic Load Allowance* suggested by Fatigue and Fracture Limit State [20].

APPENDIX C

Work Plan for the Testing of Demand Responsive Transverse Rumble Strips at the University of Nevada Las Vegas

Introduction

The Nevada Department of Transportation (NDOT) through its Stewardship and Oversight Agreement [24] with Federal Highway Administration (FHWA) has accepted the responsibility of approving experimental products for facilities located within their right-of-way. The details can be found in appendix A of the agreement, PROJECT ACTION RESPONSIBILITY MATRIX; Approve the use of proprietary products, processes. This work plan has been prepared in accordance with FHWA's experimental work plans [25]. As such, this document serves as the proposed Work Plan for the testing of Demand Responsive Transverse Rumble Strips (DRTRS) on Harmon Avenue in front of the Lied library at the University of Nevada Las Vegas. This location was chosen because it currently provides opportunities for traffic safety improvement. The location was discussed UNLV's Planning and Construction.

Description of the Experimental Feature

Figure 1 illustrates the proposed DRTRS in the field. The device is installed upstream of crossings or unsafe areas at a Stopping Sight Distance (SSD) which is the length required for vehicles to safely stop. The rumble effect is created by a set of rumble units illustrated in Figure 2. These units include three hydraulic actuators to lower or raise C Channel beams. Figure 3 presents a zoomed view of the most important components of the rumble strip units including a hydraulic actuator, the C Channel beam, a support column, and a base plate. The rumble units are separated by spacer sections made of structural steel angles and filled with concrete to provide stiffness and stability. Rumble strips and spacer sections are bolted to a steel box frame, which will be attached to the road using studs and epoxy. Rumble strips and spacer units can be disassembled separately without the need to remove the box frame. Hydraulic lines connect the actuator to a hydraulic pump and control unit that will be placed in an appropriate box on the side of the road.

The default position of the rumble strips is to have the upper surface of the C Channels flush with the road, spacer units, and the upper edges of the box frame. The rumble strips can be activated by either pedestrian push buttons, traffic signal controller, and/or vehicle/pedestrian detection systems. When a signal is sent to the system, the hydraulic actuators will lower the rumble strips C Channels; the resulting recesses create the transverse rumble strip effect.

The proposed modular design is compact, less than 6-inches deep, allowing to place it within asphalt without the need for added support or preparation. The rumble units are four inches wide. They are spaced eight inches from each other and are transverse to the flow of vehicular traffic. The C Channel beams create a rumble of 0.5 inches deep. These dimensions were chosen based on the existing literature about conventional transverse rumble strips. Different jurisdictions use different dimensions. The ones chosen for the DRTRS are consistent with most jurisdictions. However, the spacing and depth of the DRTRS can be changed relatively easy. The design is robust with relatively few components. It was developed to maintain functionality under various conditions including severe temperature variations, rain, snow, and dirt. The use of hydraulic power enhances safety because no electric lines will be used. The DRTRS can be quickly installed and uninstalled. Support columns are added to carry the load caused by the tires of the passing vehicles when the strips are at the recessed position.

Figure 20: Schematic of the DRTRS in the Field

Key components of the DRTRS include:

8) C-channel beams.
9) Hydraulic actuators that lowers the C Channel beam to create the rumble effect. These actuators are spring-loaded, which means that hydraulic power will be needed only to lower the strips.
10) A controller unit that allows various modes of input to the rumble strips.

General Operations Description

At the default position, the C Channels of the rumble strip units are flush with the top of the steel box frame and roadway. To lower the rumble units, the hydraulic actuators retract, lowering the C Channel beams. To return to the default position, the hydraulic pressure is released, allowing springs within the hydraulic actuators to push the C Channel beams up causing them to become flush with the top of the box and roadway. The DRTRS can be activated through communication with pedestrians, traffic signal controllers, and/or detection systems.

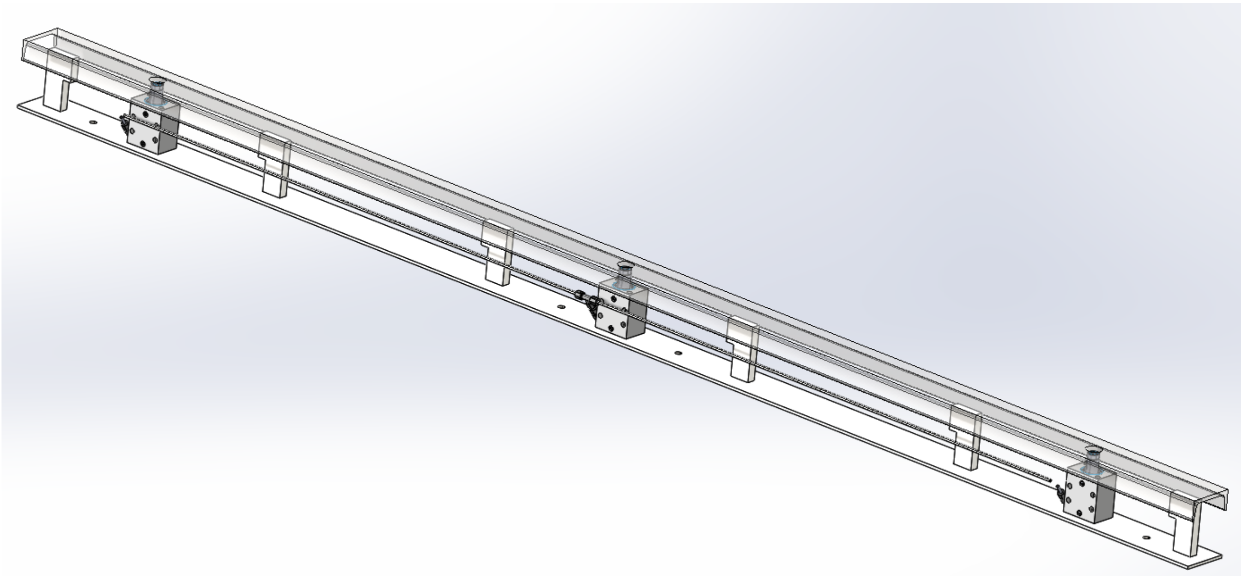


Figure 21: A Unit Box Beam and Roller Assembly of the Demand-Responsive Rumble Strips

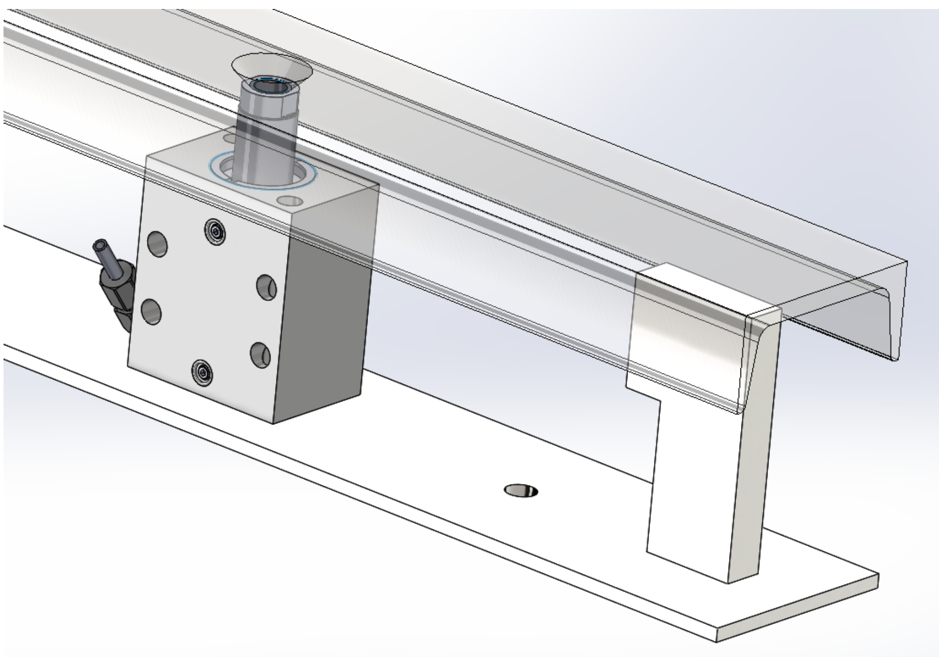


Figure 22: A Zoomed View Showing one of the hydraulic units and support of the of the Demand-Responsive Rumble Strips

Experimental Feature Objectives or Anticipated Benefits of the Product

The DRTRS would be installed on travel lanes upstream of locations with traffic safety concerns or potential for safety improvements to alert drivers/vehicles about the presence of downstream conflict. By making the mechanism active only when needed, the proposed design avoids the problem of getting drivers accustomed to the rumble strip effects while minimizing noise and vehicle deterioration as compared to permanent rumble strips. Hence, drivers' attention will be regained to address distractions, low visibility, or fatigue. The anticipated benefits of the DRTRS include reduction of the number of downstream crashes.

The DRTRS provides redundancy in the case of autonomous vehicles to minimize the likelihood of crashes due to failures and/or malfunction in the detection or navigation systems. That is, the DRTRS provides an alternative communication mechanism to alert the autonomous vehicle to slow down or stop just as it does with human drivers. The DRTRS has the potential to address at least one of the six research priorities in the Fixing America's Surface Transportation (FAST) Act; Promoting Safety.

Data to be Collected

Testing of the DRTRS will be performed using three sequential sets of experiments where different measures will be made as follows. The first two sets of experiments are currently being performed at UNLV. The third set of experiments are part of this proposed Work Plan:

1. Laboratory testing

The objective of this testing is to evaluate performance and reliability. Measurements to be made include:

i. Power required to activate each component unit and the entire system
ii. Response time
iii. Rumble position error
iv. Drainage capacity provided by the system
v. Percentage of water that enters the assembly relative to the flow of water on top of the concrete box and top plate

2. Off-the-road field testing

This testing is currently conducted at a UNLV gated lot next to the Willian D. Taylor All. UNLV vehicles including a truck owned by the Civil Engineering Department are being used. This testing is focused on durability, operational reliability, noise and vibration produced by the DRTRS. Measurements being made include:

i. Effect of traffic on the structural integrity of the DRTRS; this can be inspecting the unit regularly.
ii. Effect of traffic on the mechanical and electrical components of the DRTRS; this can be inspecting the unit regularly.
iii. Effect of sediments and small particles that can go inside the unit box beam and roller assembly without affecting operations; this can be measured by volume and by inspecting components regularly.
iv. Noise generated by the DRTRS inside the vehicles using smartphones.
v. Noise generated by the DRTRS on the road. This will be measured using smartphones.
vi. Vibration generated by the DRTRS inside the vehicles using accelerometers.
vi. Deflection generated by the design axle.

3. On-the-road field testing

This is the field test on Harmon in front of the Lied library. The following measures will be made for two months before and after installation of the DRTRS:

i. Speed of traffic measure using a video tracking system.
ii. Deceleration rates measured using a video tracking system.
iii. Noise generated by the DRTRS inside some vehicles operated by the research team. This noise can be measured using smartphones.
iv. Noise generated by the DRTRS on the road using smartphones.
v. Number of conflicts (close calls) and crashes measured using surrogate statistics such as time to collision.
vi. Volumes for vehicles, pedestrians and bicyclists measure using video counting systems.
vii. Number of activations of the DRTRS.

We will continue to monitor all variables listed in the off-the-road testing stage.

Although the main objective of the DRTRS is to address driving with distractions, fatigue, low visibility, and/or under the influence of stimulants, it is very difficult and expensive to measure these events. Hence, to the extent possible, we will estimate these events and associated DRTRS effect using the measurements listed above. In addition, a survey questionnaire will be designed and used to interview the community and users about the DRTRS and its effects on distracted driving.

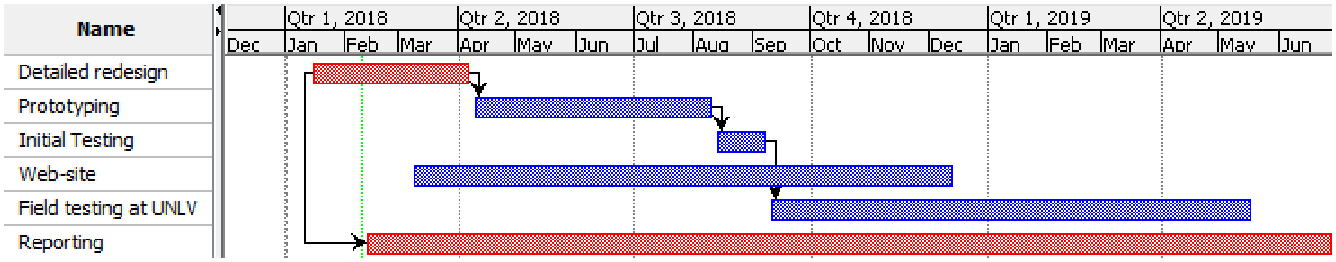
The data collected before and after the DRTRS installation will be analyzed using appropriate statistical methods to extract as much meaningful conclusions and insights as possible. Various hypotheses will be tested to assess DRTRS effectiveness. For example, our current design enables setting the depth of the rumbles at two different levels. One hypothesis is that different facilities with different geometric and operational characteristics may require a different configuration of the rumble strips to maximize effectiveness. Similarly, the DRTRS enables turn on and off various strips to increase or decrease the spacing between rumbles. These capabilities provide a large number of alternative configurations that can be tested and used in the field according to specific site characteristics. Various configurations will be setup and tested to collect as much data as possible. Systematic analysis of all these data requires special purpose statistical tools such as data count and zero inflated models to seek interdependencies among potential dependent and explanatory variables. In addition, no-parametric models will be estimated to study corrections among the various data items collected during the field testing and to determine which configuration of DRTRS provides the best benefits for different site conditions and characteristics. For example, the analysis could reveal that for the same site a different configuration is required during day and night time conditions.

Characteristics to be Evaluated

The primary objective of the DRTRS is to reduce the number of crashes as a consequence of high speeds, distracted driving, low visibility, fatigue, and/or driving under the influence of substances. Considering that crashes are rare events, the effectiveness of the DRTRS will be evaluated by comparing the before and after deceleration rates and time to collision estimates.

Time Schedules

The following table provides the list of tasks and the corresponding schedule.



Reporting Requirements

Written reports will be submitted monthly including:

1. Details of tasks performed
2. Details of data collected
3. Analysis of the data
4. Problems or challenges faced
5. Plans for the following months

Control Sections

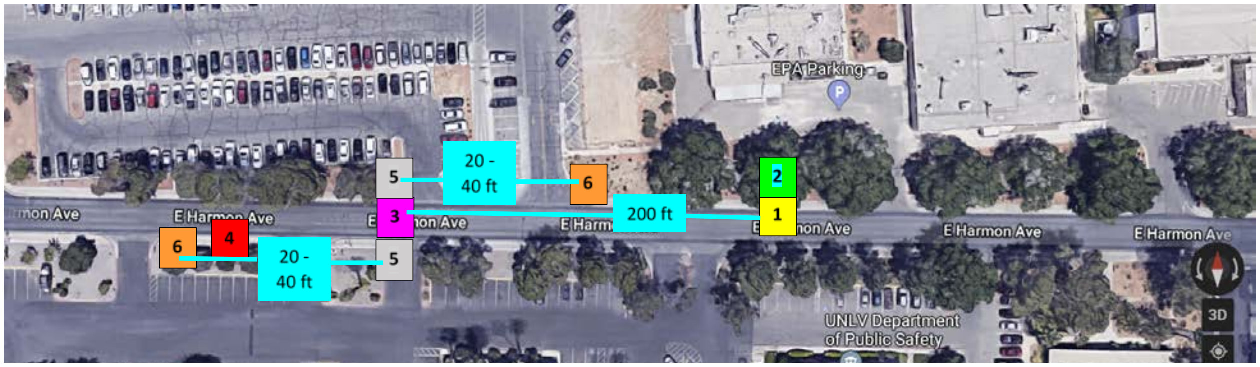
Considering that the focus and objective of the proposed feature is to reduce the number of crashes at sites with safety concerns or potential for safety improvements, we propose to use a "before and after" approach to test the effectiveness of the DRTRS and to collect the proposed measurements. Hence, the control sections will be the same locations where the DRTRS will be installed. The same type of data and measures will be collected at these sections three months before and after the DRTRS are installed.

Evaluations to be Conducted

The DRTRS will be evaluated using deceleration rates and time to collision estimates measured before and after the device is installed. In addition, the durability and maintenance required will be evaluated using strain gauges and volume of debris accumulated over time. A log of system failures or malfunctioning will be created.

Location of Proposed Installation

The proposed location for field testing is on Harmon eastbound in front of the Lied library at UNLV. The picture below depicts the approximated location of the DRTRS which will be installed upstream of a crosswalk at a distance sufficient for vehicles to slow down and stop before the crosswalk. This distance is known as Stopping Sight Distance and there is an engineering equation available in the literature for its calculation.



1) Rumble device
2) Control Cabinet (we need power here)
3) Crosswalk
4) Camera (we need power here – three 110 outlets)
5) Poles with push button
6) Pedestrian crossing signs

Figure 4 Proposed location for field testing the third design at UNLV

Location of Control Section

As indicated above, we propose to use a "before and after" approach. Hence, the control section is the test site with data collected three months before the DRTRS are installed.

Creation and/or Modification of Specifications to Allow for a Proprietary Product

The proposed feature, DRTRS, is designed and built by UNLV. The DRTRS was designed and built to enable its installation and operation using off-the-shelf materials and components.

System Monitoring and Evaluation

Right after installing the DRTRS on a site, team members will observe the performance of the device as well as the effects on traffic. Any detected issues will be addressed until all of them are resolved, if any. The team will leave the site only after a few hours of normal operations are observed.

Data will be collected continuously using video. These data will be analyzed weekly to detect any potential issue and resolve it as well as to access the benefits of the DRTRS. Issues can be detected using the video stream as well as measurements from the sensors installed to collect data.

We do not anticipate that the DRTRS will require onsite inspection and maintenance more than once a year. However, during this proposed field test, each DRTRS will be inspected weekly to observe the amount of collected sediments and small particles. If the volume of debris is significant, they will be measured and removed to prevent malfunctioning. Data about the volume of sediments and small particles that accumulate over time can be used to estimate inspection and maintenance requirements.

Report of Results Following the Requirements Set in the Approved Work Plan

A final report will be prepared including analysis and results as well as actual costs of the DRTRS, plus installation. After review and approval by NDOT's Chief Road Design Engineer and manager of facilities where the DRTRS are tested, two copies of this report will be submitted to the Federal Highway Administration (FHWA), Division Office. The FHWA Division Office should forward a copy to the National Partnership Program Manager.

Buy America Requirement

The DRTRS are built by the UNLV using American steel. Copies of the certifications provided by the suppliers will be accessible.

APPENDIX D
Survey Questionnaire

UNLV
UNIVERSITY OF NEVADA LAS VEGAS

INFORMED CONSENT

TITLE OF STUDY: Redesign, prototyping and field testing of Demand Responsive Rumble Strips for Advanced and Safe Mobility

INVESTIGATOR(S): Alexander Paz, Mohamed Trabia, and Brendan Morris

Purpose of the Study
The purpose of this research is to evaluate people's opinions and attitudes toward a demand-responsive transverse rumble strip (DRTRS) device designed to alert drivers in advance about a potential collision/crash. The device will be installed on Harmon in front of the Lied library at UNLV. Drivers and pedestrians at UNLV will be asked to complete the attached survey questionnaire before and after the deployment of the DRTRS.

Procedures
If you volunteer to participate in this study, you will be asked questions about your socioeconomic characteristics, driving and transportation habits, and your option and experience regarding the DRTRS.

Cost /Compensation
You will not be compensated for your time. This survey will take approximately 10 minutes to complete.

Contact Information
If you have questions or concerns about this study you can contact Dr. Alexander Paz at apaz@unlv.edu. For questions regarding the rights of research subjects, any complaints or comments regarding the manner in which the study is being conducted you may contact the UNLV Office of Research Integrity – Human Subjects at 702-895-2794, toll free at 877-895-2794, or via email at IRB@unlv.edu.

Voluntary Participation
Your participation in this study is voluntary. You may refuse to participate in this study or in any part of this study. You may withdraw at any time without penalty or prejudice to your relations with the university. Your responses will be kept confidential and cannot be linked back to you personally.

Participant Consent:
By beginning the survey, you acknowledge that you have read this information and agree to participate in this research, with the knowledge that you are free to withdraw your participation at any time without penalty.

1. What is your age? ____________ years

2. What is your gender? ☐ Male ☐ Female ☐ Non-Binary ☐ Prefer not to say

3. With which race do you primarily identify? (*please mark ONE box*)

☐ American Indian or Alaska Native
☐ Native Hawaiian or other Pacific Islander
☐ Asian
☐ White
☐ Black or African American
☐ Some other race
☐ Hispanic, Latino, or Spanish origin

4. What was your total income over the past 12 months?

☐ Less than $10,000
☐ $10,000 – $29,999
☐ $30,000 – $49,999
☐ $50,000 – $69,999
☐ $70,000 – $89,999
☐ $90,000 – $150,000
☐ Greater than $150,000

5. What is your primary method of transportation ***to and from places*** (i. e., home to work, school, the store, errands)? (please choose one)

☐ Automobile ☐ Public transit (bus) ☐ Motorcycle/ Motorized scooter ☐ Bicycle ☐ Walking/by foot

6. Please consider the factors listed below and choose the answer that best applies to how you feel about the ***current*** walking or biking/travel infrastructure in the Las Vegas metro area.

I feel that:	Strongly disagree	Disagree	Neutral	Agree	Strongly agree
Posted vehicle speed is appropriate for pedestrians/bikers to remain safe					
There is an adequate amount of signage and pavement markings to remind drivers to be aware of and courteous to pedestrians and bikers					
Drivers abide by the current laws and regulations in places which are intended to keep pedestrians and bikers safe					
The likelihood of a conflict/collision between a vehicle and a pedestrian or biker is low					
The likelihood of a conflict/collision between a bus and a pedestrian or a biker is low					
Additional infrastructure or technology is required to improve pedestrian and bicycle safety					
Distracted pedestrians could also be part of the issue (texting and crossing the street)					

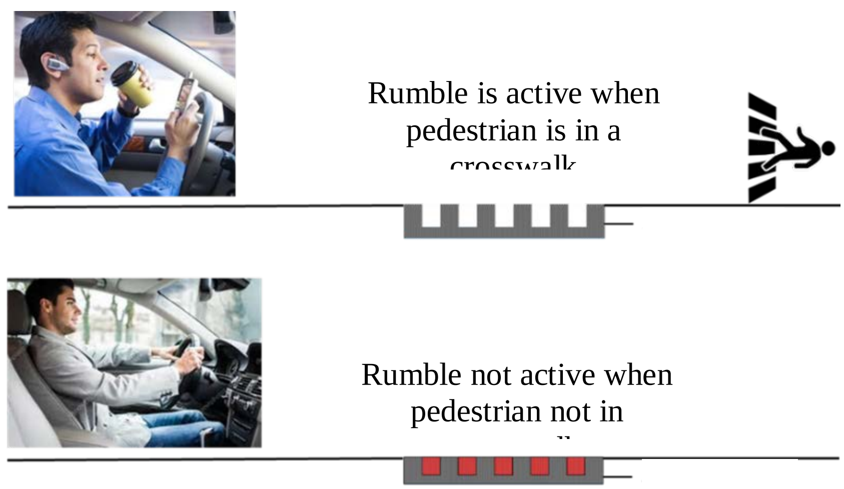
7. What are your safety concerns about walking or biking for transportation (check all that apply)?

_____ Motorists, distracted driving
_____ Too many cars/trucks
_____ Conflicts or collision with cars/trucks
_____ Conflicts or collisions with pedestrians
_____ Conflicts or collisions with other cyclists
_____ Speed of cars
_____ Potential for crime
_____ Other ____
_____ I have no safety concerns

8. How often do you:	Very often	Often	Rarely	Never
Text and drive?				
Talk and drive?				
Text and bike or walk?				
Talk and bike or walk?				
Drive over the speed limit?				
Jaywalk?				

Demand Responsive Transverse Rumble Strips (DRTRS)

A DRTRS is a new traffic safety device installed upstream of crossings or areas with safety concerns to alert drivers about potential downstream risk. The rumble effect is created by a set of rumble units that are activated on-demand to minimize noise and discomfort. The figure below illustrates the DRTRS concept.



9. Do you have experience with Demand Responsive Transverse Rumble Strip? ☐ Yes ☐ No

10. I feel that the Demand-Responsive Transverse Rumble Strips are effective to alert drivers about the presence of pedestrians and bikers.

Strongly disagree ☐ Disagree ☐ Neutral ☐ Agree ☐ Strongly agree ☐

11. How the use of Demand Responsive Transverse Rumble strips would change your willingness to walk or bike in Las Vegas?
Increase ☐ Decrease ☐ No change ☐
12. What factors would result in you starting or increasing your level of walking or biking? (check all that apply)

_____ More bike lanes

_____ Secure bicycle parking

_____ Bike lanes separated from vehicle traffic

_____ Reduced speed of cars

_____ Showers and lockers at destination

_____ Better lighting around routes

_____ More people cycling or walking

_____ The availability of a rental/shared bike

_____ Lower cost than personal vehicle commuting

_____ More bike racks on the buses

_____ Incentives from work or school (i.e.: discounted bus passes or monthly travel stipends)

_____ More information about where the bike lanes and paths are located

_____ More information about where I can access public transit (bus)

_____ More information about cost of bike and transit commuting compared to private vehicle commuting

_____ Demand-Responsive Transverse Rumble strips at crosswalks

_____ Other

____

____

13. Please list any specific investments or infrastructure changes that could be made which may result in you to walk or bike more in combination with using public transit for transportation.

____

____

____

____

14. What are your overall thoughts and concerns on the Demand-Responsive Transverse Rumble strips?

____

____

____

15. Are you willing to be contacted at a later date to provide more in-depth details of your ideas and opinions about the Demand Responsive Transverse Rumble ☐ ips? ☐ Yes
No

If yes, please provide your name and contact information below.

Name: ____

Phone: ____

Email: ____

APPENDIX E

Field Test Methodology for the First Design of the Demand Responsive Transverse Rumble Strips at the University of Nevada Las Vegas

Description

The first Prototype design was tested for both states of the DRTRS – active and inactive – with a Ford F-250 truck to have the comparative data of noise and vibration. A list of 30 runs were performed with the testing vehicle over the DRTRS inclusively without any extra load and with 2500 lbs. of loads. Among which, 15 runs while the DRTRS was active, and the rest 15 were made when the DRTRS is inactive. The variables that were measured during the test are:

Noise

- Inside vehicle noises
- DRTRS noises
- Roadside noises

Vibrations

- Inside car vibrations
- DRTRS Vibrations

Speed

A sample format for the experiment

	Sound/ Noise (dB)		Inside Car Vibration (m/s^2)			DRTRS Vibration (m/s^2)			Speed (mph)
	Inside vehicle	Outside	X	Y	Z	X	Y	Z	
Active DRTRS									
Inactive DRTRS									

Methodology for the data collection

In-vehicle noises & vibrations

Smartphone was mounted vertically on a magnetic mobile clamp close to drivers' steering. Consequently, that would provide more stability and would keep the phone fixed/ attached to the car. This system allowed us to get the actual vibration and noise data while vehicle ran over the DRTRS. The app recorded the noises in dB unit and the vibration in ms^{-2} unit for every time stamp of 0.1 second. As the smartphone was vertically mounted, the conventional 'y axis' of the smartphone served as the 'z axis' with the gravitational acceleration of 9.8 ms^{-2}.

Figure 1: data collection of Inside vehicle noise and vibration

After mounting the smartphone with the dashboard, a list of 30 runs were performed with the FORD F-250 super duty trucks with 2500 lbs. extra loads and another 30 runs without extra load. The first 15 runs of 30 (run1 to run15) were performed when the DRTRS was inactive, and the rest 15 runs (run16 to run30) were executed when the device was active. The noise and vibration for each run were recorded using the android app AndroSensor (https://play.google.com/store/apps/details?id=com.fivasim.androsensor) and were stored in the CSV format. Note that, 30 runs were performed around the speed of 25 mph.

Roadside noises

'Single vehicle pass by method' was used to collect the roadside noise data. Smartphone was mounted at 5 feet height on a tripod and 10 feet away from the edge of the DRTRS. The microphone was faced to the perpendicular direction of the DRTRS to attenuate the unwanted noises from other sources, e.g. traffic noises from the Flamingo road.

Figure 2: Roadside noise data at 10 feet from the DRTRS

Source: Field data, 2018

Smartphone has been placed on a tripod to get the roadside noise data generated by the truck while running over the DRTRS. The tripod was placed at 10 feet away from the DRTRS, and the smartphone was mounted at five feet above the ground.

DRTRS Noise and Vibrations

A smartphone was fixed with the DRTRS using the magnet that essentially yielded a steady sensor reading. Note that, the data recorded through this procedure is the true noise and vibrations of DRTRS while vehicles run over it.

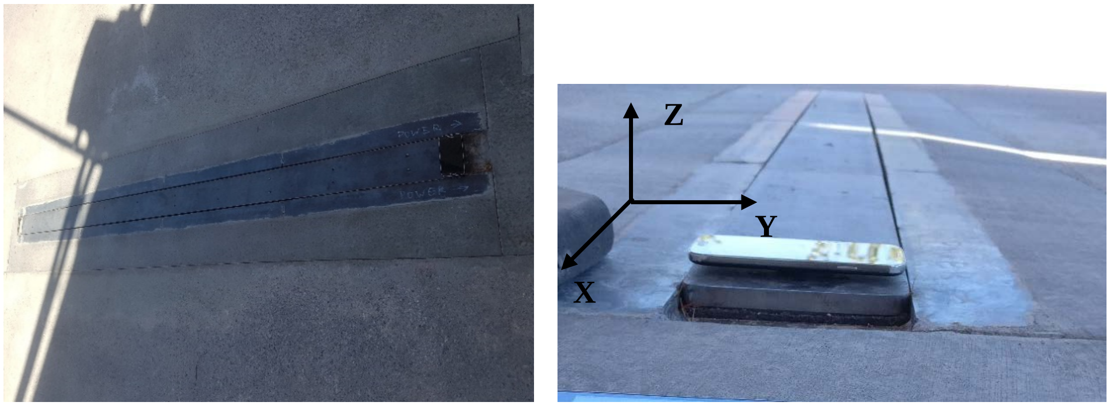


Figure 3: Noise and vibration data collection from the DRTRS

Source: Field data, 2018

X axis works here as the lateral dispersion of the DRTRS, y axis as the longitudinal ones and z axis records the vertical vibration including the gravitational unit of 9.8 ms^{-2}.

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