The IRF Examiner is an instrument designed to broadcast — and build on — the sum of academic and technical knowledge assembled during the highly successful 17th IRF World Meeting & Exhibition in Riyadh. More than 600 technical abstracts were submitted, each undergoing a rigorous peer review process, with 300 full papers ultimately being accepted, representing input from almost 60 countries. The World Meeting was also highly successful in its endeavor to encourage a multidisciplinary dialogue bridging all facets of highway development, as well as their connections with other modes of transport and the wider economy.

I would like to express my sincere thanks to the team who helped steer this project, in particular the volunteer members serving on the Review Board who assisted us in identifying the most promising contributions. I hope all of you will find the IRF Examiner a useful resource and a tangible illustration of the IRF’s commitment to better roads for a better world.

H.E. Eng. Abdullah A. Al-Mogbel
IRF Chairman
Mayor of Riyadh, Kingdom of Saudi Arabia

Global road deaths and traffic-related injuries have reached epidemic levels, causing significant personal and financial losses to society. In the United States and in 2009, there were approximately 2.2 million injuries resulting in 34,000 fatalities, approximately one third of which involved collisions with fixed objects, such as roadside trees, utility poles, steep embankments, and water hazards. There is little question that the systematic use of diagnosis tools and roadside treatment could have prevented many of these casualties.

The IRF Examiner directly supports the IRF’s stated mission of creating a global marketplace of transportation knowledge in support of informed policies and effective programs. Since our establishment in 1948, knowledge transfer has been the core of what the IRF does best. As our industry’s tools and procedures evolve to meet societies’ needs, the availability of global knowledge resources is now more important than ever.

C. Patrick Sankey
IRF President & CEO

The International Road Federation’s Road Safety Committee was formed to conduct activities to aid in the reduction of global road fatalities and serious injury accidents using various educational activities and training programs. Its programs and activities include advocating for the safe and efficient transportation of people and goods on urban and rural road systems throughout the world, promoting proper driver behavior, promoting use of appropriate education and training for road users, as well as reducing safety risks for motorists and other vulnerable road users, such as bicyclists, pedestrians, and motorcyclists.

Road safety improvements can be achieved with continued education and dissemination of best practices, new methods, and proven technologies around the world. The IRF Examiner provides one means for sharing this information with everyone and educating key people that will improve global road safety now and for years to come.

Ron Faller
IRF Road Safety Committee Chairman
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Implementing Road Safety Audits on
Local Rural Roads in South Dakota

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ABSTRACT
The fatality rate on local rural roads in South Dakota (SD) is almost 50% higher than the national average. To combat the excessive fatality rate, Road Safety Audits (RSA) were investigated to identify improvement strategies for local rural roads, which could be implemented at a relatively low cost and proactively address road safety issues before crashes occur. Rural RSA were conducted on three rural county roads, five gravel surfaced roads in Highland Township (Day County), two intersections in the City of Pierre, and two railroad crossings. Each RSA is profiled in a case study that includes a project description and summary of key findings. The aim of the case studies is to provide the SD County Highway Superintendents and local road agency managers with examples and advice assisting in implementing RSA in respective jurisdictions.

BACKGROUND
RSA were originally conceptualized and introduced in the United Kingdom in 1989 and made mandatory by 1991. The benefits of such systematic checking were soon recognized around the world and many countries have since established similar systems. Through the 1990s, audits were introduced to other countries such as Australia, New Zealand, and Canada. Audits have been conducted in the United States since late 1990. In the year 2000, Pennsylvania became the first state to formally adopt RSA into its typical processes (1). The Federal Highway Administration (FHWA) has developed a comprehensive website (http://safety.fhwa.dot.gov/rsa/) and Toolkit CD on RSA.

According to the National Cooperative Highway Research Program (NCHRP) synthesis 336, the RSA is not a means to rank or rate a project, nor is it a check of compliance with standards. In addition, the RSA does not attempt to redesign a project. Instead, it results in recommendations or findings that should be considered when a project is reviewed (2). The key elements of these definitions are as follows:

- Formal examination with a structured process and not a cursory review
- Conducted independently by professionals who are not currently involved in the project
- Completed by a team of qualified professionals representing appropriate disciplines
- Focuses solely on safety issues (2)

Road Safety Audits Benefits
Even with the budget constraints faced by most local governments, many low-cost safety countermeasures are available to address the most common and predictable crash types: run off the road rollovers and striking fixed objects. In South Dakota, these are not only the two leading types of crashes, but they account for 70 percent of deaths and injuries on local rural roads (3). When making improvements, it is often useful to think about safety
from a new point of view. Recognizing that most local road managers face increasing challenges of maintenance under difficult resource constraints, it can be productive to step back and think about safety of the roads from the perspective of those who use roads.

By conducting a RSA, an agency can improve safety and demonstrate how it is taking action to reduce crashes. A number of benefits of RSA include:

- Proactive approach to safety
- External independent perspective of safety opportunities for improvement
- Identify low-cost, high-value safety improvement opportunities
- Promote awareness of safe design and maintenance practices
- Potentially reduce costs by identifying safety issues and correcting them before projects are built
- Support requests for special safety funding (4,5)

**ROAD SAFETY AUDIT PROCESS**

The process of conducting a Road Safety Audit is managed by the RSA team leader in cooperation with the responsible road manager. According to the Federal Highway Administration (FHWA) Road Safety Audit Guidelines (6), the team leader should be an independent expert in road design, maintenance or transportation planning. The principle investigator played the role of team leader in all the RSA conducted throughout this study. The steps in the process are illustrated in Figure 1, and are discussed below with reference to the case studies.

**Project Identification**

Possible RSA project sites emerged as a result of South Dakota Local Transportation Assistance Program (SDLTAP) communications and commitments from South Dakota’s local agencies. Representative sites were selected based on highway functional classification (major collector, minor collector, or local), pavement type (gravel or paved), and intersection type (at grade highway intersections or railroad-highway crossings). The selected projects represent county paved and gravel roads, township roads, city intersections and railroad crossings.

The study included three county roads in Deuel, Lawrence, and Day Counties. Two were gravel roads, while the Day County highway was paved. The study also performed RSA at five different locations in Highland Township (Day County) on local gravel surfaced roads, two intersections in Pierre, SD, and two railroad crossings. Each of the sites had its own characteristic and presented a model case of safety features and problems.

**Team Selection**

The RSA team members were selected for each case study according to the nature and location of the project. In all cases, the teams reflected different types of expertise, including someone familiar with the road being reviewed (i.e. a school bus driver, a mail delivery person, a law enforcement officer, a road maintainer/blade operator, a truck driver, etc.), someone with multidisciplinary experience, and at least one member with professional experience in design, traffic operations, and safety, and familiar with design standards and the Manual on Uniform Traffic Control Devices (7).

The team leader was a SDLTAP staff member familiar with the RSA process who was responsible for coordinating the review, facilitating team communications, and preparing the written documentation. With the cooperation of the local road manager, the team leader selected members, coordinated calendars, and notified the team of field review dates. While the team was three to five members in size, its actual composition varied according to the focus and expectations of the review, as defined by the local road manager.

**RSA CASE STUDY PROGRAM**

Research teams conducted multiple Road Safety Audits involving twelve sites along county highways, city streets, township roads, and other locations deemed appropriate for the study. RSA projects were selected through SDLTAP promotion and commitments from South Dakota’s local agencies. The research team and the SDLTAP staff encouraged commitments from local agencies to host RSA during the study. Road Safety Audits were promoted as a useful way to identify needed low cost safety improvements. Also, RSA audits were promoted...
at county and township association meetings and at 29 safety workshops across the state. RSA team members were selected in each case study according to the nature and location of the project. The eight RSA conducted in this case study program are summarized in Table 1 (8).

### TABLE 1 Road Safety Audit Case Study Projects

<table>
<thead>
<tr>
<th>Facility Owner</th>
<th>Project</th>
<th>Case No.</th>
<th>Surfacing Type</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deuel County</td>
<td>Clear Lake Rodeo Route</td>
<td>1</td>
<td>Gravel</td>
<td>Special event traffic Development occurring, increased traffic</td>
</tr>
<tr>
<td>Lawrence County</td>
<td>Maitland Road</td>
<td>6</td>
<td>Gravel</td>
<td>Sight distance; traffic speed, six fatalities in recent years.</td>
</tr>
<tr>
<td>Day County</td>
<td>Day County Route 1</td>
<td>7</td>
<td>Gravel</td>
<td></td>
</tr>
<tr>
<td>Highland Township (Day County)</td>
<td>Township Road System</td>
<td>8</td>
<td>Gravel</td>
<td>Crest vertical curves and drainage</td>
</tr>
<tr>
<td>City of Pierre</td>
<td>Euclid &amp; 4th Street</td>
<td>3</td>
<td>Paved</td>
<td>Day care center access conflicts</td>
</tr>
<tr>
<td>City of Pierre</td>
<td>Harrison &amp; Church</td>
<td>4</td>
<td>Paved</td>
<td>Pedestrian safety</td>
</tr>
<tr>
<td>Stanley County</td>
<td>Bad River Road / DM&amp;E Road</td>
<td>2</td>
<td>Gravel</td>
<td>Skewed crossing</td>
</tr>
<tr>
<td>City of Pierre</td>
<td>Pierre Street Railroad Underpass</td>
<td>5</td>
<td>Paved</td>
<td>Low vertical clearance</td>
</tr>
</tbody>
</table>

### CASE STUDIES FINDINGS

The research team conducted several Road Safety Audits involving twelve sites along county highways, city streets, township roads, and other locations. The teams identified safety issues on most of the local roads where RSAs were conducted. Below, explanations of each safety issue and the possible increase in accident risk are briefly reported.

**Crystal Springs Rodeo Route – Deuel Co.**

This RSA addressed concerns about safety and operation related to the annual Crystal Springs Rodeo. Attendance is estimated at 10,000 over 3 days. Daily traffic volume was less than 100 vehicles per day, but estimated to exceed 2000 automobiles and heavy vehicles during the event. The road has a posted speed limit of 50 miles per hour (mph). The surface is asphalt concrete for the first mile, then gravel for four miles to the rodeo site.

Both Deuel County Highway Department and Clear Lake Township were commended for cooperative extra maintenance efforts of mowing prior to and performing daily blade maintenance during the event to safely accommodate rodeo traffic. Water is also hauled and applied to the road if dry conditions exist during the rodeo. Gravel surfaces at the time of this review were in good condition with an observable crown, no excess windrows and no secondary ditch at the edge of roadway. The review team recommends permanent traffic signing, temporary signing and delineation during event, and Vegetation removal to improve roadside safety and sight distance to be continued in the future.

**Day County Route 1**

Day County Route 1 (447th Avenue) is a bituminous-surfaces major collector running south from the southeast corner of Waubay. The section selected for review was approximately two miles in length, beginning approximately four miles south of town. The surface was in very good condition and appeared to have been chip sealed within the last couple of years. Pavement markings were in good condition, including striped no passing zones. The posted speed limit is 55 mph.

This road not only serves local access and agricultural traffic, but also acts as recreational access to area lakes. A boat ramp and parking area for Bitter Lake lie within the project limits. Both the county and township have gravel pits adjacent to the road requiring gravel hauling trucks to enter, creating potential conflicts.

At the time of review, the team did not have historical records of reportable crashes. However, there have been a total of nine fatalities in 2006 and 2008 (six within the limits of this project review). Although there are no recent traffic counts, traffic volume is increasing. The Day County highway superintendent estimated Average Daily Traffic (ADT) at 400+. Safety is a growing concern due to traffic speed and limited sight distance due to the rolling terrain (both horizontal and vertical curves).

Due to the narrow roadway and curving alignment with several roadside obstructions or drop-offs, there are several locations where enhanced delineation could contribute to improving safety. Delineators used through curves are effective guidance devices, especially at night or in bad weather when pavement markings are not visible. There were locations on reverse curves where Chevrons could be added to emphasize and guide drivers through changing horizontal alignment. As a supplement to advance curve warning signs, Chevron Alignment signs in view of the motorist can effectively define the direction and sharpness of the curve. Also there are locations along the route where entering driveways have culvert ends with steep slopes creating an unsafe obstacle adjacent to the roadway.
Flattening these slopes, and in some cases extending culverts, would improve safety for vehicles departing the roadway.

City Of Pierre Euclid and 4th Street Intersection
The purpose of the review was to address concerns about safety and operational issues related to vehicles picking up and dropping off children at the day care center located at the northeast quadrant of the intersection. The primary safety concern occurs during the morning and evening when drop-offs and pick-ups occur during the heaviest traffic periods.

Because of the volume of traffic observed approaching from the east on 4th Street, consideration should be given to re-striping the approach for a left turn lane. This would require removing the existing center line stripe and shifting it to the south to make room for a left turn only lane (minimum 9 ft.) and a through and right turn lane (minimum 11 ft.). In addition additional parking spaces on the south side will need to be removed. Also all pavement markings in the intersection should be reviewed and kept in good condition.

Bad River Road and DM&E Railroad Crossing
The primary safety concern is the potential for crashes at the railroad crossing due to the skewed angle of the crossing. The gravel-surfaced county road intersects the railroad track at a skewed angle making visibility in both directions difficult.

Vegetation was present in the northeast corner, which partially obstructs sight for any westbound vehicle looking for a west bound train. This is compounded by the skew angle forcing the driver to look back to the right. This vegetation appeared to be mostly in the railroad right-of-way, and should be removed. Another alternative discussed by the team included discussing with the DM&E Railroad the possibility of adding a train actuated flashing light on the advance warning signs. Approaching trains would activate a flashing light to give additional warning to approaching motorists.

ROAD SAFETY AUDITS FOLLOW UP AND FEEDBACK RESULTS
Shortly after the RSA were completed in May 2009, numerous actions were taken to improve safety. Positive responses included installation of new signs, upgrades to current signs, speed limit changes, and long-term plans for roadway realignment. Below, feedback to each RSA case is briefly reported.

Crystal Springs Rodeo Route – Deuel Co.
Following the report recommendations from the RSA conducted in late August 2008, Deuel County added signs and a temporary reduced speed limit during the annual rodeo. No crashes were reported during the event. Also in response to one of the RSA recommendations, Deuel County requested assistance in obtaining accurate traffic counts during the three day rodeo, as well as counts two days prior to and one day after the event. The counts will help to support decisions on any potential future safety improvements. The county highway superintendent expressed appreciation for the RSA, which armed him with recommendations from outside experts to enhance local road safety.

Day County Route 1
In June of 2009, the county commission passed a resolution requesting funding assistance from the FHWA or other source, through the South Dakota Department of Transportation (SDDOT) to make construction and operational improvements on Day County Route 1. The RSA conducted there on April 9, 2009, the final report, and the addendum submitted by the SDDOT Region Traffic Engineer served as the primary support documents for this request.

City Of Pierre Euclid and 4th Street Intersection
With the help of SDDOT, the city of Pierre’s Safety Committee redesigned the day care entrance and exit. The new design changed the main building access from the front to the rear. New “yield to pedestrians in crosswalk” signs have been placed in each leg of the intersection. These improvements were recommendations from the RSA conducted in August 2008.

Bad River Road & DM&E Railroad Crossing
Stanley County officials installed new Advanced Warning Railroad Crossing signs approximately 500 ft. from each side of the crossing. At the writing of this report, they were attempting to contact the area railroad foreman to resolve tree trimming in the railroad right of way to improve sight distance. The 35 mph speed limit has been extended approximately 1000 feet south to reduce speed northbound before entering the curve approaching the crossing.

CONCLUSIONS
- An ongoing commitment to conducting RSA on local rural roads will assist local agencies in identifying and prioritizing safety improvements. The audits can be used to implement plans that improve highway safety.
- The RSA case studies sponsored by the FHWA through SDDOT have been well received by
participating highway agencies. Shortly after the RSAs were completed, numerous actions were taken to improve safety.

- The project exposed local governments to the concept and practices of RSA and provided a good opportunity for local highway agencies and staff members to participate and gain experience from working with road safety teams.
- The major issues identified by the audits related to intersection sight distance, angle of approach, signage, road alignment, vertical and horizontal curvature, culverts, table drains, and signs location, visibility, and legibility.
- Many of the recommended solutions related to maintenance practices (e.g. keeping the immediate roadside clear of vegetation to improve sight distance through curves), delxneation (e.g. marking changes in horizontal alignment, culverts, and bridge ends with object markers), and general sign improvement (e.g. installing signs to warn of particularly sharp or unexpected changes in horizontal or vertical alignment).
- RSA offer local managers an opportunity to improve road safety by proactively addressing potential crash areas and at-risk elements not typically identified through other safety review processes. RSA can be used as a tool to plan and implement safety improvements as money becomes available.

REFERENCES

5. Wilson, F.R. and Hildebrand, E.D., Road Safety Audits for New Brunswick, Presentation to New Brunswick Department of Transportation, Fredericton, New Brunswick, April 9, 1999
An In-depth Evaluation and Analysis of Traffic and Work Zones During Highway Construction

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ABSTRACT
Analysis of construction data from New York State Department of Transportation (NYSDOT) from 1993-98 and the spatial and temporal analysis of crashes at work zones from Utah Department of Transportation (UDOT) from 2002-03 reveal that excessive speed was contributing to intrusion and transitional areas accidents at the ends of work zones, which are the most crash-populated areas. Australian incident management revealed delays in a 43-km route under reconstruction were minimized and responses to traffic incidents were reduced through an incident management center. Speed limit research was done based on the Study of the National Cooperative Highway Research Program (NCHRP) 3-41, indicating that a 16 km/h reduction in the work zone speed limit was most effective in maximizing work zone traffic safety.

INCIDENT MANAGEMENT
Work zone environment traffic movement is constantly evolving and incidents occur on semi-regular intervals. For a mid-size to large highway construction project, highway agencies will greatly benefit from partnering with a centralized traffic management system (Figure 1a and 1b) to maintain a more efficient traffic flow. The main function of such a system would be to rapidly detect traffic incidents and trigger all communication channels to clear any obstacles resulting from the incident, provide timely and accurate information to the roadway users and restore traffic flow safely and quickly.

In South East Queensland (SEQ), Australia, between Brisbane and the Gold Coast, a heavily traveled interurban route carrying up to 90,000 vehicles per day was in reconstruction, upgrading an existing 43-km from four to eight lanes and demonstrated very successful incident management. The objectives of the incident management system were to maintain smooth and safe traffic flow as well as to reduce delays and costs to the users. Planning was comprised of a number of operating standards including keeping two lanes in each direction open, mandatory speed limit of 80 km/h and the use of temporary concrete barriers.

FIGURE 1: Project overall traffic management system (1)

1.1 Conceptual operation of the traffic management system
The project planners strictly followed performance targets and developed key indicators to monitor effectiveness of incident management. The indicators include a travel time target of less than ten additional minutes of commute time through the work zone, no increase in monthly incidents impacting traffic, a reduction in incident clearance time and provision of traffic information with a target of sustained public access. Other successful strategies include the partnership plan between transportation agencies, law enforcement, fire and rescue agencies, construction contractors, towing contractors and the incident management in SEQ, which provided a catalyst for increased incident management techniques as part of traffic operations in the future. (1)

WORK ZONE ACCIDENTS

Intrusion Accidents
An intrusion accident is the entrance of a vehicle into a defined workspace, buffer space or in the transition area inside the channelizing devices. A NYSDOT construction project conducted from 1993-98 revealed a total of 290 intrusion accidents recorded and categorized as full intrusion, buffer, moving operations and access accidents (Figure 2a). Debris intrusion from debris in the traffic lanes, or a traffic control device at the workspace boundary, is thrown into the workspace by a passing vehicle, striking a worker or construction vehicle/equipment.

The NYSDOT study also reported objects struck (Figure 2b) by an intruding vehicle or debris and were categorized as contractor vehicle-equipment, physical feature-overturn-no collision, truck mounted attenuators and vehicle arrows boards, and workers and pedestrians struck by vehicles or debris thrown from passing vehicles. Other features involved within the workspace are police vehicles (marked patrol vehicles), set-up and removal of a temporary traffic control zone, and penetrated portable concrete barriers (2).

A few of the key factors provided by the NYSDOT contributing to intrusion accidents include impairment, visibility, weather, traffic control, excessive speed, and aggressive drivers. In 70 cases of the study excessive or inappropriate speed was a factor. Intrusion accidents accounted for roughly 9% of all traffic accidents, 8% of hospital injuries and 7% of the fatalities. Overall, intrusion accidents make up a relatively small number of the work zone accidents and appear to be somewhat less severe than work zone traffic accidents in general (2).

Crash Rate Analysis
Crash rate analysis refers to the analyzing of crash rates and other crash data from one or more work zones. Jin and Saito (5) conducted an in depth study of two Utah highway work zones: U.S. Route 6 (US 6) and Interstate 15 (I-15). The former highway, being a principal route in connecting some of Utah’s cities, was under reconstruction.
and rehabilitation from April '02 until August '03 with a work zone length of 3.72 miles using plastic barrel barriers. The I-15 work zone south of Nephi, Utah was under reconstruction and rehabilitation from April '02 until June '03 with a length of 11.1 miles using Jersey Barriers (5).

After in depth spatial and temporal analysis of the two projects crash records, researchers found that areas most prone to crashes consisted of the transitional areas at the end of the work zone activity. Additional analysis of the data was performed to determine which phase of construction correlated with the highest crash rates. For both US 6 and I-15 the second phase of construction was determined to be the most dangerous. The study also recorded multiple other variables for which crash rate data could be recorded by phase. The following conditions and variable factors were found to have similar outcomes by phase (5):

- Light condition
- Traffic control
- Weather conditions
- Surface conditions

The similar outcomes from both work zones indicate what factors should be accounted for with a focus on traffic safety planning and accident prevention plans being formed. The study showed that although different types of traffic control devices (barriers) were being used on both work zones, there were still enough similarities in the data to outline which areas were most accident prone. The results highlight the transitional areas and suggest that more time be spent on these areas in which the data expressed to be of greater significance (5).

WORK ZONE SPEED LIMITS

A procedure for determining work zone speed limits is presented in the NCHRP Project 3-41. While the National Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD) presents uniform guidelines for determining the speed limit on roads free of work activities, it does not provide a comparable procedure for determining work zone speed limits (3).

Accident data, from before and during work activities, was collected for stationary construction and maintenance zones. A minimum work period of one month was established for accident study sites with duration of work at most sites being substantially longer. Speed data were collected upstream of and within work zones during daytime off-peak, daytime peak, and nighttime periods. Work zones were classified by highway type, urban or rural location of work activities, and work zone traffic control treatments. The speed parameters were as follows: mean speed, 85th percentile speed, speed limit compliance and speed variance (3).

For work zones with speed limits that were not reduced, the speed variance in the work zone was 61% higher than the upstream speed variance. For work zones with a 16 km/h speed limit reduction, the speed variance in the work zone was only 34%. Finally, for work zones with a speed limit reduction of 24 km/h or more, the increase in the work zone speed variance above the speed variance upstream of work zones was in the range of 81-93%. The differences between the mean percentage increase in the accident rate for a speed limit reduction of 16 km/h and other reductions were statistically significant. Work zones without speed reductions had the next smallest percentage increase in the fatal-plus-injury accident rate (3).

The data suggests that when work zone speed limits are established using the engineering factors of the NCHRP work zone speed limit procedure, drivers reduce their driving speeds about the amount of the reduction in the posted speed limit. The NCHRP work zone speed limit procedure provides a rational method that is applicable to stationary construction zones, maintenance zones, and utility operations, as well as intermittent moving or mobile operations; and continuous moving operations on all streets and highways in urban and rural areas at speed limits up to and including 105 km/h (65 mph). The recommended procedure has four steps (3):

- Determine the existing speed limit
- Determine the work zone condition that applies
- Determine which factors for the appropriate condition apply to the specific site
- Select the work zone speed limit reduction

To assist with reduction of speed in the work zone, research was done at the University of Kansas to provide assistance in the optimal deployment location of Portable Changing Message Sign (PCMS) upstream of work zones. It was concluded that the optimal location of the PCMS is anywhere from 556 feet to 575 feet away from the first W20-1 sign (‘Road Work Ahead’) upstream of the work zone (6).

A survey of work zone speed limit policies in the United States was conducted by Migletz et al (3) and the responses received from different states presented in Table 1. The majority of states reduce the speed limit in work zones based on various procedures or factors (3).
TABLE 1: State Work Zone Speed Limit Policies (3)

<table>
<thead>
<tr>
<th>States that avoid reducing work zone speed limits whenever possible (29%)</th>
<th>States with “blanket” reduced work zone speed limits (8%)</th>
<th>States that reduce work zone speed limits based on an identified procedure or set of factors (63%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alaska&lt;br&gt;Arkansas&lt;br&gt;Connecticut&lt;br&gt;District of Columbia&lt;br&gt;Iowa a,c&lt;br&gt;Kansas a,c&lt;br&gt;Maine</td>
<td>Louisiana&lt;br&gt;Michigan&lt;br&gt;Montana a&lt;br&gt;Puerto Rico a</td>
<td>Alabama a&lt;br&gt;Arizona&lt;br&gt;California b,c&lt;br&gt;Colorado&lt;br&gt;Delaware&lt;br&gt;Florida b,c&lt;br&gt;Georgia b,c&lt;br&gt;Hawaii&lt;br&gt;Idaho&lt;br&gt;Illinois&lt;br&gt;Indiana</td>
</tr>
</tbody>
</table>

a States whose work zone speed limit policies have changed since the NCHRP Project 3-41 survey.
b States that participated in Project 3-41, the first study.
c States that participated in Project 3-41, the second study.

APPLICATION OF EXISTING PREDICTION TOOLS

Evaluation of Models Applied to Work Zones

The Ohio Department of Transportation (ODOT) uses various existing prediction tools to calibrate data obtained from four work zones on multi-lane freeways. Data was gathered using traffic flow video recording vans. Traffic flow parameters were extracted from the video records using the Mobilizer-PC (M-PC) software package. The traffic simulation and prediction tools included the Highway Capacity Software (HCS), Synchro, CORSIM, NetSim, and a macroscopic model called QueWZ92. Simulation models were constructed with all models for the selected work zones, and the simulated queue lengths and delay times were compared to the data that was extracted from the field data with M-PC (4).

Data Collection Procedure

Four work zones in the state of Ohio were selected by ODOT. Video footage was recorded by two ODOT video recording vans (Figure 3). The first van filmed traffic within the work zone lane taper. The second van filmed traffic free flow conditions several miles upstream from the work zone. Spotters also observed and manually recorded the queue length every ten minutes throughout the formation and dissipation of queues. These queue length measurements were accomplished by a trained spotter in a vehicle that was moving back and forth with the queue, but on the opposite side of the freeway. Accurate queue length measurements were thereby obtained (4). It is always recommended to obtain the free-flow traffic volume during a 24-hour weekday period and a 24-hour weekend period before initial planning of a work zone.

FIGURE 3: Mobilizer-PC capturing traffic data for the analysis (4)

Video van and camera boom set up for Mobilizer-PC analysis

Screen shot of the Mobilizer-PC tracking procedure
Model Application
The M-PC traffic flow analysis program was used to analyze the video records from the work zone. The M-PC produced parameters such as speed, headway, vehicle type, and density. In addition, queue lengths and delay times were observed in the field. Analysis of the data provided confirmation that QueWZ92 provided the optimal predictions of the work zone capacity. Also the program was specifically designed for work zones, generating fairly accurate capacity estimates.

QueWZ92 is able to make delay times (queue delay and speed delay) predictions on freeway or multi-lane divided highways containing up to six lanes for twenty-four hour periods. In work zones, users need to specify the lane closure configuration (i.e., three lanes to two lanes), schedule of work activity, and traffic volume. QueWZ92 includes a series of constants, such as percentage of heavy trucks, a cost update factor, speed-volume relationship, and the numerical definition of excessive queuing. The program does not allow for factors such as the width of the open-lane and grade (4).

ODOT developed a spreadsheet to estimate the queue lengths along a one-lane freeway work-zone using Quattro Pro. The spreadsheet allows for a simple and quick estimation of the queue length with minimal user inputs and is able to estimate the queue length based on both the observed and the estimated volumes. The work zone capacity estimate can be obtained using QueWZ92 and is used by the ODOT spreadsheet to estimate the queue length (4).

CONCLUSIONS
Incident management can be readily applied utilizing a project traffic management center equipped with appropriate technology by observing traffic flow upstream, downstream and in the work zone. Application of technologies such as variable message signs, motorist assistance programs, emergency services, and public awareness will help to establish a safer work zone. In addition, a comprehensive communication system that shares information among project teams, road users, local agencies, police, and fire/rescue workers will help develop optimal inter-agency cooperation.

Portable concrete barriers are very effective in preventing intrusions. Accidents during temporary traffic control set-up and removal accounted for 8% of the total incidents. These incidents point out the importance of proper layout of the temporary traffic control zones (2).

Crash rate analysis research discovered that crash rates are highest in transition areas of the work zones in varying conditions clarifying the need that the areas near the beginning and end of the work zones are in need of the highest attention when it comes to managing the work zone to create a safer work zone environment.

Implementation of a work zone speed limit is essential for safety. The procedure developed in NCHRP Project 3-41 is implementable and can be used to set speed limits that maximize work zone traffic safety. The application of work zone speed limits results in safer and more efficient traffic operations (3).

The most precise tool in predicting the capacity of a work zone is QueWZ92. The most accurate tool in estimating the maximum queue lengths proved to be the ODOT spreadsheet when provided with an adequate capacity estimate such as that from QueWZ92. Tables exist tabulating work zone capacities based on the vehicle mix, grade, work zone length, lane width, and lane closure configuration. These tables were developed by the ODOT and can be used easily to get the needed information for estimating the maximum expected queue lengths in a work zone environment (4).

A comprehensive and in-depth evaluation and analysis of traffic and work zones during highway construction is needed. ‘An In-Depth Evaluation and Analysis of Traffic and Work Zone During Highway Construction’ has focused on compiling dispersed information from various sources in an orderly fashion for the user.
REFERENCES


Development of a Cycle-Based Red-Light Violation Index For Signalized Intersections

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ABSTRACT
Departments of Transportation (DOTs) at state, county and city levels in the U.S. and equivalent global agencies have been implementing diverse methods including signal timing optimization, geometric improvements, media campaigns, and Red-Light Running Cameras (RLRC) for reducing red light violations (RLV) and crashes at signalized intersections. RLV rates based on traffic exposure and crashes at signalized intersections are often included among factors for making decisions about RLRC installations. The research investigates the development of the background red light violation index (RLVI) based on the probability that a signal cycle would experience a RLV by vehicles that would have an opportunity to run the red light. The cycle-based red light violation index (CRLVI) model developed in the research relied on data collected from multiple signalized intersections in Washington, D.C. (DC).

WASHINGTON, D.C. RED LIGHT RUNNING ANALYSIS AND MODELING
Between 1999 and 2010, the DC Metropolitan Police Department installed 52 red-light cameras (RLC). The installations were in response to a perceived high rate of RLV at intersections, and a need to mitigate the RLV problem. There is no established basis in DC for determining when the RLV experience of an intersection is beyond expectation. A set of criteria is essential to enable the City to objectively prioritize the implementation of strategies for reducing RLV, including engineering and public awareness programs. Several measures are documented on red light running (RLR) issues. Issues include the number of violations per certain volume of vehicles, number of violations per unit time, number of violations per cycle, as well as complex algorithms for calculating a RLV rate.

The Red Light Violation Problem
From 1990 through 2007, numerous cases point to the fact that on average, 25% of crashes at intersections were crossing path crashes, associated with RLR [4,5,6,7,8]. A 2007 report by the National Highway Traffic Safety Administration (NHTSA) suggested that intersection crossing path crashes account for approximately 25% of all police-reported crashes in the United States each year [6]. They also account for 27% of all crash-related delays and over $47 billion in costs [6].

Factors Related to Red-Light Running
Factors contributing to red light running include vehicle characteristics, intersection design and operation, weather, and driver behavior [9]. Vehicle characteristics may be linked to inappropriate signal timing since vehicle characteristics are factored into any traffic signal design. Deficiencies in traffic signal design and the geometrical configuration of signalized intersections may contribute to unintended RLR behavior.
A 2006 NHTSA report indicated that a decrease in RLR was found to be associated with the following factors [1]:

- Decrease in approach flow rate
- Increase in yellow duration
- Decrease in speed
- Increase in clearance path length (i.e. a wider intersection)
- Decrease in platoon density
- Addition of signal head back plates

To combat the problem of violations, when suspected to be frequent, the NHTSA report recommended to confirm the extent of the problem through the computation of the expected frequency of red-light running for the subject location and to compute a “ranking index” that indicates whether the site is truly a problem location [1].

**Agency Approach to Resolving Red-Light Running**

Crash and accident reduction factors provide a quick way of estimating crash reductions associated with highway safety improvements, and are used by many states and local jurisdictions in deciding whether to implement specific treatments and/or to quickly determine the costs and benefits of selected alternatives [11]. Arising out of the analysis of crash and accident reduction factors is accident modification factors, which are then implemented by the respective agencies, and may include such items as improvements to intersection geometry and signal timing. The research method required analysis after identifying an issue of RLV, and making an assessment of whether the RLV was excessive or not, determining which recommendations for improvement of the intersection could be made [12].

The Federal Highway Administration (FHWA) and the Institute of Transportation Engineers [13] suggested that while intentional violators will likely be most affected by law enforcement countermeasures, unintentional violations could be addressed, in part, by engineering countermeasures, including:

- Improving signal visibility
- Improving signal conspicuity
- Increasing likelihood of stopping
- Addressing intentional violations
- Eliminating the need to stop

**Red-Light-Violation Prediction Models for Countermeasure Implementation**

The Red-Light-Running Handbook, developed by Bonneson and Zimmerman (2004) for the Texas Department of Transportation, suggests guidelines for identifying and treating locations that have an unusually large number of RLV or related crashes [14]. The method employed the Texas RLR Evaluation and Analysis Tool (TREAT), to identify problems of red light related safety, based on crash data for an intersection, approach or jurisdiction and making a comparison with the average annual crash frequency of similar intersection approaches. If the subject approach had significantly more crashes than similar approaches it was identified as having potential benefit from treatment. For local intersections, the analysis would be based on the observed frequency of RLV [14]. Data for evaluating the RLV problem, according to the TREAT method, include:

- Traffic characteristics
- Traffic control
- Signal operation
- Motorist information
- Traffic operation
- Geometry
- RLV analysis time period
- Crash history

If the index developed was determined to be negative, then the intersection would be described as operating at or below the expected RLV frequency. If the index was positive, then the intersection would be operating above the expected RLV frequency, and countermeasures could be recommended based on the degree of positivity [14].

**Impact of Various Agency Approaches to Curb Red-Light Running**

McGhee et al., 2002 summarized research findings from a number of studies on the effect of RLC on crash occurrence and severity. Based on a comparison of data, some authors were of the opinion that there is no conclusive evidence that suggested RLC prevent RLR crashes [15]. The researchers found that when rear-end collisions were taken into consideration, there were minor reductions in both property damage only accidents and injury crashes. Thus establishing that there was a significant side effect associated with the installation of RLC such that the investment may not return the expected benefits. Compounding the problem is overestimation of RLV at intersections, where practitioners neglect to adjust for regression to the mean (Retting, Ferguson, and Hakkert, 2002) [16].

In DC, statistics have shown an overall decrease of 75% in total RLV during the installation period 1999 to 2007 [3]. However, a closer inspection of the statistics revealed that in some cases the violation rate remained about the same, or was actually higher. While the reasons may be unknown, it lends credence to the literature that the installation of RLCs is not the only solution to the problem of RLR at intersections.
DATA COMPILATION AND REDUCTION

Data was collected at one approach of the 18 intersections using a video camera for a two-hour period during the morning off-peak period between 10:00 AM and 12:00 PM where potential for RLR was high. Off-peak periods were deliberately selected in this research in order to minimize RLV that are associated with poor traffic operating conditions. Selection of the approach for videotaping was based on volume and having adequate vantage location for positioning of the video team and camera. Playback was used to extract information for vehicles per hour of green, duration of green, clearance distance, and lane configuration. Posted speed limits on the approaches were obtained during site visits.

Vehicles per Hour Green of Approach
Vehicles per hour green of interval approach are defined as the number of vehicles served by green interval for approach to the intersection under study including all the vehicles crossing the stop bar on green, while excluding those vehicles that are considered violators under the definition. This variable was chosen on the theory that the percentage of vehicles utilized by the green compared to the actual number of vehicles traversing the intersection in a given cycle, may lead drivers to run the red light. Literature suggests that this variable directly correlates with RLV at urban intersections.

Duration of Green
Duration of green refers to the actual green time allocated to an approach at each intersection. Duration of green affects queue build-up at an intersection, which impacts the potential for red light running.

Clearance Distance
Travel distance from the stop bar to the edge of the far side of the intersection was used as a variable in the analysis. Literature suggests that the wider the intersection, the less likely a motorist would be to run a red light, assuming signal timing and all other variables were constant. In the case of DC intersections, the duration of yellow for all intersections at the time of study was 4 seconds. Therefore, it was theorized that for DC intersections, a direct and significant correlation could be made between this variable and red-light-running.

Lane Configuration
The lane configuration variable relates to the number and type of lanes at the intersection approach, as well as number of turns. Each through lane at an intersection was given a code of 1, and each turning lane was given a code of 0.5. Values were summed for the total of each type of lane and the value used in the analysis and combination of these lanes is considered an independent variable in the model.

Posted Speed Limit
The posted speed limit on approach to the intersection was included in the analysis. This variable was chosen as several reports quoted in literature having found a relationship between RLR and the posted speed limit within an area. Effects of the posted speed limit can be such that if drivers believe it to be inadequate for the particular zone, users may drive above the posted limit. This then has implications for RLR, as driving above the speed limit has been quoted as the major factor in drivers being caught in the dilemma zone. In cases where there was a variable posted speed, for example, in the vicinity of a school, the upper speed limit was chosen, which reflected the condition at the time data was collected.

Establishing Background CRLV Probability Model
In the development of the CRLVI, each independent variable was tested against the dependent variable, cycle red light violation probability (CRLVP) to determine the degree of correlation. The following generalized regression model was determined to be adequate:

$$CRLVI = \alpha + \beta_1 G + \beta_2 V + \beta_3 V^3 + \beta_4 W + \beta_5 W^2 + \beta_6 L + \beta_7 L^3 + \beta_8 S + \beta_9 S^2 + \beta_{10} S^3 + \epsilon,$$

- CRLVI = Cycle Red-Light Violation Index (same as the background CRLVP)
- G = Duration of Green
- V = Vehicles per Hour Green of Approach
- W = Clearance Distance
- L = Lane Configuration
- S = Posted Speed Limit

The constants $\alpha$ and $\beta$ are the coefficients of the regression model with an associated error of $\epsilon \sim N(0, \sigma^2)$. The statistical significance of the regression coefficients were tested at 5% level of significance. Similarly, the overall statistical significance of the regression model for the intersections was tested using the F-test at 5% level of significance. The F-test tests the significance of the overall model by determining if the variance accounted by the model was reasonably large. If the associated $p$-value of the F-test was less than 5%, then the regression model would be acceptable and that the hypothesis of the non-existence of a relationship between the independent variable and the dependent variable would be rejected. The regression model was also checked for homoscedasticity (constant variance) using residual plots, and also checked for normality using the normal probability plots.

Upon obtaining the regression model, an analysis of the predicted values from the model was compared to the observed probabilities for RLR to determine whether there
was a statistically significant difference between the two data sets. This was done using the Kolmogorov-Smirnov test, which provides a D statistic, indicating the difference between the data sets. If the D statistic was less than the expected D statistic for the sample size, and normality was obtained, then it could be said that there was a goodness of fit between the predicted and observed values for RLV.

**Characteristics of Intersections**
The selected intersections are located in the DC urban environment. All intersections were signalized consisting of a cycle length of 120 seconds and a four second yellow interval. The intersections and data extracted are contained in Tables 1 and 2:

**TABLE 1: Intersection Characteristics**

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Intersection Name</th>
<th>No. of Legs</th>
<th>Observed Approach</th>
<th>No. of Lanes</th>
<th>Parking Traffic</th>
<th>Pedestrian Traffic</th>
<th>Heavy Vehicles</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>New Mexico Avenue @ Westover Place NW</td>
<td>3</td>
<td>SB</td>
<td>1</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>2</td>
<td>Pennsylvania Avenue @ 8th Street SE</td>
<td>4</td>
<td>WB</td>
<td>4</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>5</td>
<td>Michigan Avenue @ 7th Street NE</td>
<td>4</td>
<td>EB</td>
<td>2</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>6</td>
<td>MLK Junior Avenue @ Malcolm X Avenue SE</td>
<td>4</td>
<td>NB</td>
<td>2</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>8</td>
<td>Connecticut Avenue @ Newark Street</td>
<td>4</td>
<td>SB</td>
<td>3</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>10</td>
<td>Connecticut Avenue @ Devonshire NW</td>
<td>3</td>
<td>SB</td>
<td>3</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>11</td>
<td>Benning Road @ 32nd Street NE</td>
<td>3</td>
<td>EB</td>
<td>4</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td>12</td>
<td>13th Street NW @ N Street NW</td>
<td>4</td>
<td>SB</td>
<td>3</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>13</td>
<td>13th Street NE @ Maryland Avenue NE</td>
<td>4</td>
<td>NB</td>
<td>2</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td>14</td>
<td>6th Street SE @ Constitution Avenue</td>
<td>4</td>
<td>NB</td>
<td>2</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>15</td>
<td>34th Street @ Benning Road N</td>
<td>4</td>
<td>WB</td>
<td>5</td>
<td>No</td>
<td>No</td>
<td>Yes</td>
</tr>
<tr>
<td>16</td>
<td>K Street NE @ 7th Street NE</td>
<td>4</td>
<td>EB</td>
<td>2</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>17</td>
<td>Franklin Street NE @ 14th Street NE</td>
<td>4</td>
<td>WB</td>
<td>2</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
</tr>
<tr>
<td>18</td>
<td>North Capitol Street NE @ Gallatin Street</td>
<td>4</td>
<td>NB</td>
<td>3</td>
<td>No</td>
<td>Yes</td>
<td>Yes</td>
</tr>
</tbody>
</table>

**TABLE 2: List of Surveyed Intersections and Reduced Data**

<table>
<thead>
<tr>
<th>Intersection</th>
<th>Vehicles Per Hour Green</th>
<th>Green (secs)</th>
<th>Clearance distance (ft)</th>
<th>Approach Lane Configuration</th>
<th>Posted Speed Limit (mph)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>319</td>
<td>66</td>
<td>92</td>
<td>2.5</td>
<td>25</td>
</tr>
<tr>
<td>2</td>
<td>657</td>
<td>59</td>
<td>90</td>
<td>3.5</td>
<td>30</td>
</tr>
<tr>
<td>3</td>
<td>586</td>
<td>49</td>
<td>121</td>
<td>4.0</td>
<td>25</td>
</tr>
<tr>
<td>4</td>
<td>593</td>
<td>34</td>
<td>130</td>
<td>2.5</td>
<td>30</td>
</tr>
<tr>
<td>5</td>
<td>536</td>
<td>52</td>
<td>143</td>
<td>3.0</td>
<td>25</td>
</tr>
<tr>
<td>6</td>
<td>341</td>
<td>51</td>
<td>103</td>
<td>3.0</td>
<td>25</td>
</tr>
<tr>
<td>7</td>
<td>536</td>
<td>64</td>
<td>90</td>
<td>2.5</td>
<td>30</td>
</tr>
<tr>
<td>8</td>
<td>1014</td>
<td>63</td>
<td>100</td>
<td>4.0</td>
<td>25</td>
</tr>
<tr>
<td>9</td>
<td>659</td>
<td>40</td>
<td>113</td>
<td>3.0</td>
<td>25</td>
</tr>
<tr>
<td>10</td>
<td>817</td>
<td>61</td>
<td>85</td>
<td>4.0</td>
<td>25</td>
</tr>
<tr>
<td>11</td>
<td>693</td>
<td>66</td>
<td>68</td>
<td>5.0</td>
<td>30</td>
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<tr>
<td>12</td>
<td>382</td>
<td>55</td>
<td>102</td>
<td>2.5</td>
<td>25</td>
</tr>
<tr>
<td>13</td>
<td>196</td>
<td>26</td>
<td>118</td>
<td>2.0</td>
<td>25</td>
</tr>
<tr>
<td>14</td>
<td>308</td>
<td>55</td>
<td>122</td>
<td>1.5</td>
<td>25</td>
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<tr>
<td>15</td>
<td>900</td>
<td>67</td>
<td>68</td>
<td>4.5</td>
<td>15</td>
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<tr>
<td>16</td>
<td>180</td>
<td>24</td>
<td>72</td>
<td>1.5</td>
<td>25</td>
</tr>
<tr>
<td>17</td>
<td>158</td>
<td>46</td>
<td>70</td>
<td>1.5</td>
<td>25</td>
</tr>
<tr>
<td>18</td>
<td>727</td>
<td>72</td>
<td>66</td>
<td>3.5</td>
<td>25</td>
</tr>
</tbody>
</table>
RESULTS

A statistically significant relationship ($F = 0.011$) was found between the selected independent variables and the dependent variable, CRLVP. The CRLVP, based on the number of cycles with violations, divided by the total number of observed cycles, was computed for each intersection, and recorded in Table 3.

<table>
<thead>
<tr>
<th>Intersection No.</th>
<th>Cycles</th>
<th>Cycles with Violations</th>
<th>Cycle Violation Probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>81</td>
<td>6</td>
<td>0.074</td>
</tr>
<tr>
<td>2</td>
<td>88</td>
<td>9</td>
<td>0.102</td>
</tr>
<tr>
<td>3</td>
<td>84</td>
<td>33</td>
<td>0.393</td>
</tr>
<tr>
<td>4</td>
<td>80</td>
<td>20</td>
<td>0.250</td>
</tr>
<tr>
<td>5</td>
<td>74</td>
<td>6</td>
<td>0.077</td>
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<td>6</td>
<td>70</td>
<td>9</td>
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<td>0.240</td>
</tr>
<tr>
<td>9</td>
<td>88</td>
<td>22</td>
<td>0.250</td>
</tr>
<tr>
<td>10</td>
<td>71</td>
<td>17</td>
<td>0.239</td>
</tr>
<tr>
<td>11</td>
<td>73</td>
<td>11</td>
<td>0.151</td>
</tr>
<tr>
<td>12</td>
<td>73</td>
<td>0</td>
<td>0.000</td>
</tr>
<tr>
<td>13</td>
<td>55</td>
<td>5</td>
<td>0.091</td>
</tr>
<tr>
<td>14</td>
<td>47</td>
<td>11</td>
<td>0.234</td>
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<td>15</td>
<td>57</td>
<td>15</td>
<td>0.263</td>
</tr>
<tr>
<td>16</td>
<td>93</td>
<td>0</td>
<td>0.000</td>
</tr>
<tr>
<td>17</td>
<td>57</td>
<td>2</td>
<td>0.035</td>
</tr>
<tr>
<td>18</td>
<td>53</td>
<td>7</td>
<td>0.132</td>
</tr>
</tbody>
</table>

The percentage of variance ($R^2$) explained by this model was 89%, suggesting that the majority of the causes of RLR in the DC area could be explained by the independent variables. The summary results for the Analysis of Variance (ANOVA) are presented in Table 4.

![FIGURE 1: Kolmogorov-Smirnov Comparison Test Plot Showing the Differences Between the Actual and Predicted CRLV Rates for the Study Intersections.](image)

![FIGURE 2: Normal Probability Plot of CRLVP vs. Sample Percentile](image)

DISCUSSION OF RESULTS

The results showed for the 18 intersections studied the relationship between the CRLVI and the independent variables proved statistically significant, at 95% confidence interval. The amount of variance explained by the independent variables amounted to 89%. The results of the preliminary model test indicated that the formula developed has shown promise as a tool for predicting the CRLV probability for intersections in DC during the off-
peak period. The differences between the observed and the predicted values were generally within one standard deviation of the observed values.

CONCLUSIONS

The typical cycle violation probability for DC is 0.1636. For every 100 traffic signal cycles, approximately 16 of those will experience red light violations. The analysis shows that the intersection-related variables are strong predictors of RLR in DC. Given that the percentage of variance explained by the model was as high as 89%, it means that modifications to the engineering properties studied in this research may lead to a significant reduction in RLV in the District. The research suggests that CRLV in DC can be predicted with reasonable accuracy. Further study is required to enhance the model so it could become applicable to a broader configuration of intersections. The model is only applicable to intersections in dense urban areas with characteristics similar to behavior of those studied. The CRLVI developed in this research, suggests that a reliable model for characterizing the level of RLV for at-grade signalized intersections is achievable. Conceptually, the model has broad practical appeal since it relies on engineering factors that are easily obtained at signalized intersections. The research advances the state of knowledge on using the probability of red light violated cycles as a measure of the status of RLV at intersections and its potential for use in determining the effectiveness of implemented RLV projects and programs.

REFERENCES


School Zone Safety in Abu Dhabi

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ABSTRACT

The purpose of the School Safety Zone Project (SSZP) initiated by the Municipality of Abu Dhabi City (ADM) is to make schools zones safer for children in Abu Dhabi City. The project implements engineering measures to provide drivers with early indication of schools in the vicinity (red zoning and signage), reduced speeds (traffic calming), enhanced pedestrian connectivity at crossing points around schools (speed tables and zebra crossings), and rationalization of parking habits and drop offs (lay-bys). Approximately 240 schools under the stewardship of the Abu Dhabi Education Council (ADEC) were identified for treatment under the project. Phase 1 of the project safety measures were implemented in 2012 for 44 schools within the mainland and island. To enable the implementation of the project, Abu Dhabi Safety and Traffic Solutions committee team formed in 2013, which included nominated members of key stakeholders.

SCHOOL SAFETY ZONE, WHY THE NEED?

The SSZP is integral to a wider Abu Dhabi Safety initiative wherein Municipality of Abu Dhabi City carried out the following:

- Network wide risk assessment (2700km of road network assessed)
- Island wide safety improvements initiated
- Implementation of road safety program
- Implementation of SSZP
- Public awareness initiative

The team developed to carry out the initiative consisted of the Traffic Police, Abu Dhabi Education Council (ADEC), Department of Transport and Town Planning and Road Safety Unit at ADM. The roles and responsibility of representatives in the team would be to review and finalize the preliminary design concepts for the proposed safety measures and implementation of best industry practice for the remaining 200 schools.

The schools have been split into 6 packages and implementation would be rolled out package-wise. The team conducted meetings at the schools to introduce the project and questionnaires were provided to schools in order to capture their needs and requirements. The designs prepared would incorporate the comments received from schools and feedback from site visits conducted by the ADM engineers. The finalized designs would then be passed onto the Execution Department Contractors at ADM for implementation.

The project construction phase would commence after schools closed for summer holidays and completed before the new academic year starting September 1, 2013. New schools under construction that would be open for the academic year 2013-2014 were also included.

Parents, teachers and members of the community often identify vehicle speeds along routes to school as a concern. Research shows there is good reason for these fears, as the chances a pedestrian will survive a crash with a vehicle decline rapidly the faster the car was driving. Because of the direct relationship between vehicle speed and severity of injury, traffic calming measures and speed limit enforcement, all aimed at slowing vehicles, are common objectives of the SSZP.

The population of Abu Dhabi is largely made up from expatriate communities and over 200 nationalities with varied driver behaviours, add to that the wide roads and prevalence of powerful expensive cars and opportunities to drive them fast. Unfortunately many drivers drive at speeds much over the allowed speed limits within sector roads where schools are located.

Most traffic accidents are the product of several factors. The probability of accidents can be reduced in a number of different ways. Statistically the main cause of death among children 0-14 year olds is fatal injuries, of which 63% are traffic related (2).

Although there are no statistics for accidents near School Zones, the team has gathered inputs of accidents due to speeding near schools including some nearly fatal. Children under the age of 14 are the most over represented group in pedestrian crashes and vulnerable, thus it becomes important to provide adequate safety facilities in the vicinity of land uses such as school zones.
Most of the above can be attributed to the following:

- Limited use of school buses, 40% of students use buses while 60% use private cars
- Inadequate parking spaces for the large number of private cars
- Lack of designated drop-off or pick-up areas
- Lack of supervised/controlled drop-off and pick up of students
- In some cases, poorly designed school layouts
- Unavailability of shaded walk routes.

There are three main approaches to preventing accidents:

- Provide education and training to children in schools by road-traffic instructors and teachers as well as communicating the message to the parents and drivers on the principles of safe driving attitudes near schools. Media campaign and other publicity to create awareness about the School Safety Zone Program and to draw the attention of all road users both to dangers and to safe practices implemented near the schools.
- Enforcement by adopting reasonable and enforceable traffic laws which, at the same time, are best designed to prevent speeding and accidents during school hours; enforcing speed limits near school safety zones and penalizing violators endangering children's safety.
- Engineering by improving existing roads by realignment of parking and circulation, improving connectivity to enhance pedestrian movement, speed calming techniques, road marking (red zoning) and signage like School Zone and other regulatory signs such as ‘Stop’ and ‘Give Way’ signs.

Under this project only engineering aspects would be applied to enhance safety around school areas and provide better connectivity and safe drop-off and pickups during school times.

**PROCESS & STAKE HOLDERS**

Phase 2 of the SSZP commenced in January 2013 with the formation of the team under the guidance of the Safety and Traffic Solutions Committee. The inter-agency collaboration was required to fast track the process of approvals and roll out before the commencement of the next academic year 2013-14. The schools were split into 6 packages to cover all areas in Abu Dhabi.

The team included nominated members from:

- Traffic Services Section
- Abu Dhabi Traffic Police
- Abu Dhabi Education Council
- Department of Transport
- Town Planning
- Traffic and Road Safety Unit at ADM

<table>
<thead>
<tr>
<th>PACKAGE #</th>
<th>AREAS COVERED</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Baniyas East &amp; West and Al Wathba</td>
</tr>
<tr>
<td>2</td>
<td>Khalifa City A, B &amp; Shawamekh</td>
</tr>
<tr>
<td>3</td>
<td>Mohammed Bin Zayed City, Mussaffah and Officer’s City</td>
</tr>
<tr>
<td>4</td>
<td>Shahama, Al Bahia, Al Rahba, Samha and Al Falah Areas</td>
</tr>
<tr>
<td>5</td>
<td>Abu Dhabi Island East</td>
</tr>
<tr>
<td>6</td>
<td>Abu Dhabi Island West</td>
</tr>
</tbody>
</table>

Under the guidance of ADEC, the team conducted workshops and presentations for schools to introduce the project and capturing the requirements by distributing questionnaires. Visits and surveys of all schools where team members interacted with the school management were used to identify key issues, verify on-site conditions, and used both drop-off and pick-up times when the congestion is at the peak to consider while drawing up measures. Post receipt of the questionnaires and site visits, ADM would prepare concepts of improvement for each package, which would then be passed on to other stakeholders to review and finally town planning approvals to roll out to the execution phase.

**ENGINEERING MEASURES TO BE IMPLEMENTED**

The following measures would be implemented as a part of the solutions in each school:

**SIGNAGE**

School zone traffic signs are a combination of regulatory and warning signs. Each has its own warrants and typical placement location within the school zone vicinity and would be provided. All signage would be installed as per ADMUTCD standard guidelines.

- **Children**
  - Warning sign posted wherever children activity might occur. In school zones a supplementary plate would be posted on roads in school zone vicinity at an advance distance as per ADMUTCD standards.

- **Vertical Ramps or Speed Hump**
  - A warning signs that alert drivers that one or more speed
humps exist in the roadway ahead and that driver needs to reduce speed in order to negotiate them.

**Pedestrian Crossing Ahead**

Sign warns drivers of vehicles that there is a marked and/or signalized pedestrian crossing ahead at which pedestrians have right-of-way. A Supplementary Plate sign indicating the distance to the pedestrian crossing should be attached as sign 427.

**Give Way to Pedestrian**

Regulatory signs by which the driver of a vehicle should yield right-of-way to pedestrians crossing the roadway were introduced. Give Way to Pedestrians sign 303 should be located on the right side of the roadway 3 meters in advance of Give Way pavement marking 602. Give Way pavement marking 602 should be at least 3 meters and preferably 6 meters in advance of Pedestrian Crossing pavement marking 603.

**School Zone Speed Limit**

A regulatory sign by which a speed limit of 30 km/hr is enforced throughout the school zone was introduced. School Zone is defined as the entire length of the school compound facing the zone and, at a maximum, up to 150 meters past the school boundary in either direction.

**School Bus Drop-off / Pickup**

Drop-Off / Pick-up Only zones were introduced for designated parking areas reserved exclusively for school buses.

**School Bus Parking**

A regulatory sign that prohibits the driver of any vehicle (other than school bus) from stopping his vehicle at any time along the section of road beyond such a sign.

**ROAD MARKING**

**School Zone Gateway**

In the school zone areas there shall be a gateway to inform the drivers that they are entering a School Zone area.

**SPEED CALMING**

Before any speed table or speed hump there should be a sign warning the drivers. For speed tables there should be an additional crossing sign.

**Speed Table**

Speed tables located in the pedestrian crossing areas allow connection as close as possible to the school gate.

**Speed Humps**

Speed humps located in the areas where the traffic should slow down especially on the carriageway and parking areas.

**DROP-OFF AND COLLECTION**

**School Buses Drop Off and Collection Areas**

Buses drop off areas should be safe for students and separated from car movement.

**Parents Drop Off / Collection Area**

Parents drop off areas should be near the school gate.

**PARKING**

The Project will also provide approximately 50 additional parking spaces around the schools where land is allocated to schools creating additional capacity. The project team would also endeavor to separate the buses from the cars within the proposed parking areas. In addition, geometry and circulation would be studied and changes addressed accordingly.

**CURRENT LAYOUT AND PROPOSED CHANGES**

Figure 1a and 1b show both the current layout and proposed changes that include the engineering elements previously introduced. In this concept design, the movement of buses and cars in the parking has been...
separated and connectivity to the main gate has been provided.

**CONCLUSION AND LESSONS LEARNED**

Although most schools have been constructed and are growing in capacity with increased traffic congestion and pose a risk to children walking to school during pick up and drop off, simple and basic traffic engineering measures proposed will have significant impact on driver and pedestrian behaviours in the sense that the road user is tempted and most cases tend to follow instructions or guidelines.

Each school is unique and contain independent issues to be dealt with to achieve safety initiatives and a ‘one-solution fits all approach’ is not advisable, e.g. sector ME-9 in Mohammed Bin Zayed (MBZ) City, where there are over 8 schools in one sector and 16,000 students studying in them. Here an overall traffic impact study and analysis is needed.

It is high time that to make appropriate steps in a quick manner to make the schools safe for our children and have it congestion-free, accident-free with smooth flow of traffic, which ensures safety to the public and our children.

Reduced speed limit within the school zone is the key factor to increase safety. One way traffic circulation around school is safer through geometry modification. Signage provides guidance and reduces confusion in the school zones and promotes safety. Involvement of school management during the design phase is very essential to the project success. The engineering solution is not enough to ensure school safety. Proper education and enforcement laws are also required, which would be handled by the relevant authorities.

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OPPORTUNITY

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TRINIDAD
MAY 7-9

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May 7-9, 2014
Port of Spain, Trinidad

International Road Federation

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Potential Crash Reduction Benefits of Shoulder Rumble Strips in Two-Lane Rural Highways

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Original: July 30, 2013
Revised: November 14, 2013

ABSTRACT
A study of the effectiveness of shoulder rumble strips in reducing run-off-the-road (ROR) crashes on two-lane rural highways using the Empirical Bayes (EB) Before-and-After analysis method was performed. The comprehensive procedure adopted for developing the safety performance function of EB analysis also considers the effects of roadway geometry and paved right shoulder width on the effectiveness of shoulder rumble strips. The results demonstrate the safety benefits of shoulder rumble strips in reducing the ROR crashes on two-lane rural highways using the State of Idaho 2001-2009 crash data. A 14% reduction in all ROR crashes was realized after the installation of shoulder rumble strips on 179 miles of two-lane rural highways in Idaho. The results indicate that shoulder rumble strips were most effective on roads with relatively moderate curvature and right paved shoulder width of three feet and more.

RUN-OFF-THE-ROAD CRASHES
Run-off-the-road (ROR) crashes account for a large number of severe crashes in the United States. In 2011, the total number of fatal ROR crashes was 15,307 resulting in 16,948 fatalities, which accounted for 51% of the total fatal crashes in the United States (1). A large proportion of ROR crashes occur on rural highways, especially on two-lane roads. The Federal Highway Administration (2) reports that about 70% of ROR fatalities occur on rural highways and about 90% occur on two-lane roads. The geometry of rural two-lane highways often includes sharper curve and narrower shoulder width, which increase the risk of frequency and severity of ROR crashes on rural highways. Most of the times ROR crashes involve only a single vehicle and driver performance errors, specifically distraction, drowsiness, fatigue or inattention are often identified as contributory risk factors for many of the single vehicle ROR crashes (3, 4). Rumble strips are used as a safety measure to reduce the frequency and severity of ROR crashes specific to driver performance errors.
Shoulder rumble strips are installed along the edge of the travel lane to alert drivers when their vehicles are drifting off the roadway and address the class of crashes related to driver inattention. The safety benefit of shoulder rumble

IRF Examiner
strips in reducing the frequency and severity of ROR crashes have been emphasized in many earlier studies (5, 6, 7, 8), however, the research methodologies, target roadways, and the range of results obtained in earlier studies are considerably different.

Most of the studies in transportation safety research have used the before-and-after analysis to evaluate the safety benefits of roadway treatments such as shoulder rumble strips. The objective of the before-and-after analysis is to compare the actual number of crashes that occurred after the installation of a safety measure with the expected number of crashes that would have occurred during the after period had the treatment not been installed. In this study, before period crash counts refer before the installation of the treatment and after period crash counts refer crash counts after the treatment has installed.

**DATA DESCRIPTION**

**Data Source**

The first data source is a vehicle crash report (VCR) data from Idaho Transportation Department’s (ITD) Office of Highway Safety (OHS). All law enforcement agencies in Idaho are required to send VCR forms to the OHS, who maintains and archives the data. Crash data for this study was obtained from the OHS crash database using a web based crash analysis system developed (WebCARS) and maintained by the OHS. Crash data obtained from WebCARS included the number of units involved in the crash, the mile point location selected, the date, and the first harmful event.

The second data source is from ITD office of Highway Operation and Safety (OHOS). OHOS provided all data regarding the year and the location of the installation of shoulder rumble strips examined in this study. Shoulder width, lane width and advisory speed data for the highway sections were also provided by ITD OHOS. The third data in the form of Average Annual Daily Traffic (AADT) was obtained from ITD Automatic Traffic Recorders. The fourth data source used in this study is Google Earth for the roadway sections.

**Data Assembly**

Road curvature data was obtained from both from Google Earth and speed advisory signs from ITD sign inventory database. Three different roadway curvature types for different road segments were used in the analysis: straight road segments or road segments with slight curvature, moderate curvature road segments with horizontal curves with relatively large radius, and sharp curvature segments that require significant reduction in speed.

**Treatment Sites**

Treatment sites were selected from three highways in Idaho: US 12, US 30 and US 95. From 2004-2007, shoulder rumble strips were installed along 260 mile two-lane highway segments in Idaho. After removing data for roadway segments with varying geometrics, speeds and lane configurations, 38 two-lane highway treatment sites with a total length of 179 miles of roadways were used to evaluate the safety effectiveness of shoulder rumble strips. Lane width for all test sites was a standard 12 feet. The paved right shoulder width of the test sites ranges between 1 to 7 feet. The AADT of the selected test sections varied from 500 to 7,500.

Before and after average yearly crashes per 5-mile road segments and the weighted average AADT are presented in Table 1. The before and after average yearly crashes per 5-mile road segments for the three roadway curvature types are presented in Figure 1. The average number of crashes for all treatment sites dropped from 1.341 crashes to 0.721 crashes, which showed an average 46% reduction in ROR crashes after the treatment. An analysis comparison between the before and after period crashes shows the highest reduction in ROR crashes was found for road segments with moderate to sharp horizontal curves.

<table>
<thead>
<tr>
<th>Road Curvature Type</th>
<th>Number of Sites</th>
<th>Length (mile)</th>
<th>Yearly Crash / 5 mile</th>
<th>AADT</th>
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<td></td>
<td>0.570</td>
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<tr>
<td></td>
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<td></td>
<td>-45%</td>
<td>-3%</td>
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<tr>
<td>3</td>
<td>21</td>
<td>100.66</td>
<td>1.653</td>
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<td>0.839</td>
<td>1627</td>
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<tr>
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<td></td>
<td>-49%</td>
<td>-5%</td>
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<tr>
<td>Average</td>
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<td></td>
<td>1.341</td>
<td>2460</td>
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<td></td>
<td></td>
<td></td>
<td>0.721</td>
<td>2373</td>
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<tr>
<td></td>
<td></td>
<td></td>
<td>-46%</td>
<td>-4%</td>
</tr>
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</table>
However, the comparison does not take into account the causal factors that change with time and cannot identify the reduction in ROR crashes due to installation of shoulder rumble strips. In fact, rumble strips are primarily designed to prevent inattention related ROR crashes and reduction in crashes attributed to the treatment is expected to be the most where drivers are more likely to be inattentive. This is because drivers are likely to be more attentive while driving on road sections with sharp horizontal curve and less attentive when roadways are straight. Therefore the reduction in number of crashes due to the treatment is expected to be more for road sections with less horizontal curve compared to the road sections with sharp horizontal curve.

**Control Sites**
Control sites are randomly selected from the same three US highways (i.e. US 12, US 30 and US 95): a total of 58 sites were selected for the two-lane highway before and after study. Yearly crash data for the selected 58 control sections are assembled for year 2001 to 2009. The control sites data were used to develop the safety performance function (SPF) of the EB analysis. Total length of the control sites was 277 miles with no shoulder rumble strips installed on any of them during the analyzing period. Control sites had traffic volume and geometric characteristics similar to the selected treatment sites.

**ANALYSIS AND RESULTS**

**Development of the SPF**
The SPF developed for the study using the control site data is assumed to follow an NB regression model. Generalized linear model methods were used to perform the regression analyses, using with a log link function. A log linear relationship between the mean number of crashes and the independent variables is specified by the log link function. The log link function ensures that the dependent variable of the model, which is the mean number of crashes per year per segment of a given length for the fitted model, is positive.

The independent variables considered for the model are AADT, length of road segment, right shoulder width, road curvature type, and year. Among these variables, AADT was introduced as a continuous variable, and the roadway segment length was introduced as an offset variable for which no regression parameter was estimated. The year variable was introduced to consider various unobserved factors such as demographic changes that took place throughout the duration of the study period. Right paved shoulder width, road curvature type, and the year variables were introduced as class variables.

The sign of the coefficient of the AADT variable is positive and in-line with earlier research studies (13, 19). The positive coefficient value represents that the probability of ROR crash frequency increases with increasing AADT. The coefficients of the road curvature variables are estimated where road curvature type 3 is the base category. As expected, sharper horizontal roadway curvature increases the likelihood of the higher number of ROR crashes in two-lane rural highways. Right shoulder width ≥ 7 feet is the base category for the right shoulder width variable. Compared to the base category, the signs of the coefficients of right shoulder width less than 5 feet are positive and the signs of the coefficients of right shoulder width five feet and 6 feet are negative. The positive signs for right shoulder width less than five feet indicate that the narrower shoulder width (width < 5 feet) increases the risk of higher number of ROR crashes. The risk of a ROR crash is lowest for right shoulder width of five feet followed by six feet compared to the base category.

The likelihood ratio statistics for the selected independent variables and their corresponding p-values indicate that AADT, right shoulder width, and year variables are
significant at the 0.05 level in determining the number of ROR crash for two-lane rural highways. The road curvature type variable is statistically significant at the 0.10-significance level.

**EB Analysis Results**

The observed number of ROR crashes after the installation of shoulder rumble strips was 92 and the EB estimates of the expected crashes without any treatment were 106.5. The unbiased estimate of safety effectiveness index and its variance was calculated for each test sites using the EB procedures. The unbiased estimate of the safety effectiveness index and its variance were calculated over all road sections. It was estimated that the installation of rumble strips resulted in an overall 14% reduction for all ROR crashes. The corresponding standard deviation was estimated as 11%.

Although the installation of rumble strips resulted an overall improvement in the reduction of ROR crashes, the examination of each treatment site shows the variability among the effect between the treatment sites. The variability can be attributed to a number of unobserved factors such as environmental factors (light condition, weather condition, pavement condition etc.), driver specific factors etc. The safety effect of shoulder rumble strips on different AADT range, road geometry type, and shoulder width are summarized in the Table 2.

| TABLE 2: EB Analysis Results for Different AADT Levels, Paved Right Shoulder Widths and Road Curvature Types |

<table>
<thead>
<tr>
<th>Number of Sites</th>
<th>Length (mile)</th>
<th>Count of Crashes During After Period</th>
<th>Expected Crashes During After Period Without Treatment</th>
<th>% Change in Crash</th>
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<tr>
<td></td>
<td>Before</td>
<td>After</td>
<td>Actual Counts EB Estimates</td>
<td>Standard Deviation</td>
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<tr>
<td>AADT</td>
<td></td>
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<tr>
<td>AADT</td>
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<tr>
<td>% Change in Crash</td>
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<tr>
<td>-33% (p = 0.003)</td>
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<td>-3% (p &lt;0.0001)</td>
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<tr>
<td>% Change in Crash</td>
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<td>-3% (p &lt;0.0001)</td>
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<td>100.7</td>
<td>65</td>
<td>70.32</td>
</tr>
</tbody>
</table>

**Treatment Evaluation in Context of AADT**

The EB estimated index of effectiveness shows that shoulder rumble strips were the most effective in reducing ROR crashes in low-volume road sections. Road sections with an average AADT less than 1000 showed a statistically significant estimated 33% reduction in ROR crashes after the installation of shoulder rumble strips. The result is not surprising because in low volume road sections drivers are more likely to drive less attentively and rumble strips can alert drowsy drivers when they are about to leave the road.

Two-lane highway sample road sections with moderate and high AADT showed a respective 16% and 24% in reduction in ROR crashes after shoulder rumble strip installation. Road sections with relatively high AADT
experienced a 24% reduction in ROR crashes after rumble strip installation. The results support the safety benefits of shoulder rumble strips in reducing ROR crashes are different for different road traffic volume.

**Treatment Evaluation in Context of Road Geometry Type**
The actual and expected number of ROR crashes for three different road curvature types support our hypothesis about the effect of road geometry on the effectiveness of shoulder rumble strips in reducing the ROR crashes. Installing rumble strips in two-lane rural highways resulted in 22% reduction in ROR crashes for road sections with no horizontal curve (roadway curvature type 1), 29% reduction for road sections with moderate horizontal curves (roadway curvature type 2), and 8% reduction for road sections with sharp horizontal curves (roadway curvature type 3).

All the results are statistically significant. The results indicate that shoulder rumble strips were most effective in reducing ROR crashes for roads with relatively moderate curvature (road curvature type 1 and type 2) and less effective in sections with sharp horizontal curves. As expected, drivers are more likely to be inattentive while driving in straight or moderately curvy road sections. In such road sections shoulder rumble strips can help them the most when they are about to leave the roadway.

**Treatment Evaluation in Context of Shoulder Width**
As expected the effectiveness of shoulder rumble strips in reducing ROR crashes are minimal for roadway sections having narrower paved right shoulder (width < 3 feet). Narrower right shoulder provides less recovery area beyond the shoulder and can reduce the effectiveness of the shoulder rumble strips. In such roadway sections site specific road safety measures should be evaluated. Right paved shoulder width of 3 feet and more shows a higher reduction in ROR crashes after the treatment.

Surprisingly the reduction in ROR crashes for road sections with right paved shoulder width of 5 feet shows a relatively smaller reduction compared to road sections with right paved shoulder width of 4 feet and 6 feet. The difference in treatment effect for different right shoulder can be attributed to other unobserved factors (such as different curve delineation, light condition, pavement condition etc.) that can affect the effectiveness of treatment in reducing ROR crashes. The safety effect of the treatment for the roadway section with right paved shoulder width of 7 feet is not statistically significant because of the limited number of treatment sites with right shoulder width of 7 feet and more.
REFERENCES


ABUJA
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JUNE 4-6
Analysis and Full-Scale Crash Test of a High-Containment Bridge Rail

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ABSTRACT
Bridge rails are built to safely redirect errant vehicles in order to protect occupants and prevent excessive damage to the bridge rail structures. Criteria to evaluate the performance of bridge rails are described in detail in working standards such as the National Cooperative Highway Research Program Report 350: Recommended Procedures for the Safety Performance Evaluation of Highway Features and European Norm (EN) 1317. The EN 1317 test TB 11 involves a 900-kg passenger car impacting the barrier at a speed of 100 km/h and an angle of 20 degrees. EN 1317 test TB 71 involves a 30,000-kg heavy-goods vehicle impacting the barrier at a speed of 65 km/h and an angle of 20 degrees. Subsequently, two full-scale crash tests (EN 1317 tests TB 11 and TB 71) of the bridge rail system were conducted and the rail system has passed EN 1317 criteria for containment level H4a.

CASTLE PEAK ROAD IMPROVEMENT PROJECT – BRIDGE RAIL
The Hong Kong Special Administrative Region (SAR) funded the Castle Peak Road Improvement Project, which includes construction of an 800-meter dual two-lane viaduct between Tai Lam Kok and Siu Lam. The bridge rail design was analyzed using the commercial nonlinear finite element (FE) program LS-DYNA and was verified with the performance of two full-scale crash tests (1). The impact severity level of the rail was evaluated against European Norm (EN) 1317-2 recommendations for the Acceleration Severity Index (ASI) and theoretical head impact velocity (THIV) (2,3). The simulations and testing reported in the study correspond to containment level H4a, which requires tests TB 11 and TB 71. Test TB 11 involves the 900 kg car impacting the barrier at an impact speed of 100 km/h and 20 degrees. Test TB 71 involves the 30,000 kg rigid heavy-goods vehicle (HGV) impacting the barrier at an impact speed of 65 km/h and 20 degrees.

RESEARCH APPROACH
The research methodology consisted of the following steps:
- Perform FE analysis of the proposed bridge rail design according to EN 1317 standards and modify the design as needed
- Evaluate proposed rail design performance using TB 11 and TB 71 criteria:
- TB 11 to check for rail stability and occupant impact severity through small car impact
• TB 71 to check for rail strength through HGV impact simulation
• Once a design is finalized, perform crash tests per EN 1317 TB 11 and TB 71 criteria

FINITE ELEMENT ANALYSIS

The original bridge rail design consists of a safety shape concrete parapet cast on a 0.250-meter thick bridge deck. Positive connectivity between the parapet and bridge deck is achieved using T11 starter bars spaced at 0.150 m intervals. A precast concrete panel is placed on the field side of the parapet and is connected to the parapet by a top and bottom layer of T10 bars spaced at 0.150 m intervals. A steel rail is connected to the top of the concrete parapet. The steel rail consists of two 0.200 m × 0.100 m × 0.016 m tubular steel rail members attached to fabricated steel posts by angle brackets and a steel plate. The posts are welded to a 0.400 m × 0.325 m × 0.045 m steel baseplate. The baseplate is anchored to the top of the concrete parapet by five stainless steel anchor bolts embedded in the concrete.

Vehicular Stability Analyses for Small Vehicles

The barrier stability analysis was evaluated by performing FE impact simulations using the Geo Metro small passenger car model. For this type of evaluation, the concrete barrier was modeled with rigid material behavior. The rail on top of the concrete barrier was modeled using steel rigid material properties. To capture the behavior of the vehicle after impact with the barrier model, the contact NODES_TO_SURFACE was added between the vehicle parts and the barrier components. In the FE simulation, the Geo Metro model at a speed of 100 km/h and an angle of 20 degrees impacted the original Hong Kong bridge rail design. Simulation results show that, after impact, the car was redirected by the barrier into the traffic lane with an ASI of 1.32 and an acceptable post-impact vehicle trajectory, according to the EN 1317 box criteria. The car, however, underwent a considerably high angle of roll. Thus, researchers decided to modify the barrier design to obtain better performance in terms of vehicle stability during redirection.

The original barrier design was modified into an F-shape rail profile. The same initial speed and angle conditions applied in the previous case were replicated for simulation of the Geo Metro impact against the new barrier design. Simulation results showed an acceptable and more stable car trajectory with redirection after the impact event. The ASI was 1.43. Results from FE simulations are summarized in Figure 3 in terms of post-impact vehicle trajectory. Since the F-shape barrier profile was acceptable for vehicular stability and occupant protection, the next step was to evaluate the barrier rail profile for strength.

Rail Strength Analyses for Heavy-Goods Vehicles

The barrier strength analysis was evaluated by performing FE impact simulations using the 30,000 kg HGV model impacting the barrier at 65 km/h and an angle of 20 degrees. For this type of analysis, the F-shape barrier included material details and reinforcing modeling. At the predicted impact location, barrier concrete was modeled with material MAT 159 Cap Surface Concrete Model (CSCM), which offers the option of damage detection. The concrete barrier was modeled with elastic material behavior to allow for material deformation without high computational costs. Steel rail, bolts, and rebar were all modeled with bilinear elastic-plastic material behavior. Rebar was coupled with the concrete through use of the CONstrained_LAGRANGE_IN_SOLID option. FE bridge rail model details are reported in Figure 4.

The FE simulation predicted considerable barrier deflection resulting from the impact with the HGV (Figure 5a). A high level of axial stresses was detected in the T12 modeled starter bars (Figure 5b), suggesting the barrier-to-deck connection resistance should be reviewed and...
redesigned. To increase the resistance of the barrier-to-deck connection, the starter rebar were changed to T19 (#6) type. Simulation was performed to re-evaluate the strength of the new rebar design, and results showed the barrier successfully redirected the HGV after impact (Figure 5c).

Computer simulation results recommended the F-shape barrier profile with rebar T19 type to be tested with the small passenger car and HGV under EN 1317 specifications.

FULL-SCALE CRASH TEST

Test Installation
The test installation was built to simulate the safety railing that will be constructed for a new bridge being built as part of the Castle Peak Road Improvement Project. The bridge rail consists of an F-shape concrete parapet cast on a 0.250 m-thick bridge deck. The continuous foundation beam is 0.635 m wide and 1.540 m deep, and its top surface was cast flush with the surface of the surrounding concrete runway. Positive connectivity between the parapet and bridge deck is achieved using T19 starter bars spaced at 0.150 m intervals. A precast concrete panel was placed on the field side of the parapet and is connected to the parapet by a top and bottom layer of T10 bars spaced at 0.150 m intervals. A steel rail is connected to the top of the concrete parapet. The steel rail consists of two 0.200 m × 0.100 m × 0.016 m tubular steel rail members attached to fabricated steel posts by angle brackets and steel plate. The posts are welded to a 0.400 m × 0.325 m × 0.045 m steel baseplate. The baseplate is anchored to the top of the concrete parapet by five stainless steel anchor bolts embedded in the concrete. A typical section and reinforcement of this bridge rail system are shown in Figure 6a and 6b. A 36-m test installation (Figure 6c) was constructed at the Texas A&M University Riverside Campus.

The concrete parapet was cast in place on top of the foundation beam in six 6 m-long segments. The concrete compressive strength was specified as 40 MPa. An expansion joint was provided between each adjacent segment. To add continuity across segments, four 25 mm diameter × 800 mm long stainless steel dowel bars were placed across each joint. One end of each dowel bar was embedded directly into the end of a concrete parapet segment, while the opposite end was inserted into a pipe sleeve cast into the end of the adjacent parapet segment to permit relative movement between the segments in the longitudinal direction.

A 5 m wide × 130 mm thick asphalt overlay was placed in front of the bridge parapet to simulate the pavement overlay that will be applied to the actual bridge deck and achieve the proper interaction height between the test vehicles and parapet. The asphalt was tapered to the surrounding concrete pavement on a gentle 1.7 percent grade to provide a smooth approach and exit for the test vehicles.

Test Vehicles
A 1996 Chevrolet Geo Metro was used for the small car crash test (TB 11). Test inertia weight of the vehicle was 836 kg, and its gross static weight including the test dummy was 910 kg. The height to the lower edge of the vehicle’s front bumper was 400 mm, and the height to the upper edge of the front bumper was 650 mm. The vehicle was directed into the installation using a cable reverse tow and guidance system, and was released to be freewheeling and unrestrained just prior to impact.

A 1991 ERF SP Cab Tipper vehicle was used for the HGV crash test (TB 71). The test inertia weight of the vehicle was 30,450 kg. The height to the lower edge of the vehicle’s front bumper at the corner was 476 mm, and the height to the upper edge of the front bumper was 940 mm. The vehicle was directed into the installation using a remote control guidance system, and was released to be freewheeling and unrestrained just prior to impact.

Small Passenger Car Test TB 11
The 1991 Chevrolet Metro impacted the barrier at the third construction joint at an impact speed of 102.4 km/h and an impact angle of 20.0 degrees. At approximately 0.039 s after impact, the vehicle began to redirect and was airborne at 0.184 s. At 0.220 s, the vehicle lost contact with the barrier and was traveling at an exit speed of 89.6 km/h and an exit angle of 3.8 degrees. The maximum roll angle was −15 degrees.
Damage sustained by the barrier was only cosmetic in nature. The barrier did not fracture or otherwise fail, and there were no detached parts. The ground connections/fixings remained intact. Dynamic deflection, permanent deflection, and working width were all 0.0 m.

The ASI was 1.69. The THIV was 30.9 km/h, and the post-impact head deceleration (PHD) was 11.6 g. The THIV and PHD meet the index values specified in EN 1317-2:2005.

Heavy Goods Vehicle Test TB 71
The 1991 ERF SP Cab Tipper impacted the bridge rail at an impact speed of 67.2 km/h and an angle of 19.3 degrees. At approximately 0.037 s after impact, the front of the vehicle reached the steel post downstream from the joint at impact (i.e., post 5), and at 0.044 s, the vehicle began to redirect. At 0.349 s, the vehicle was traveling parallel to the bridge rail at a speed of 60.9 km/h. At 0.578 s, the vehicle lost contact with the bridge rail and was traveling at an exit speed of 60.7 km/h and an exit angle of 1.4 degrees. The vehicle continued traveling adjacent to the bridge rail, and as the vehicle reached the end of the installation, 23.8 m downstream of the point of impact, the vehicle began to yaw counterclockwise. The vehicle subsequently came to rest 84.1 m downstream of impact and 32.3 m toward the field side of the barrier. The maximum yaw and roll angles were 25 and −17 degrees, respectively.

The concrete barrier fractured in the vicinity of steel posts 4, 5, and 6, but the posts did not detach from the installation. The upper rail element was pushed upward a maximum of 8 mm at a location 360 mm upstream of post 5. Maximum dynamic deflection of the rail elements during the test was 0.102 m. There was no permanent deflection of the concrete barrier. Lower and upper rail element deformations were 22 and 64 mm, respectively. The working width was 0.589 m. Although not required for test TB 71, impact severity values were computed and are reported for information purposes. The ASI was 0.66. The THIV was 19.9 km/h, and the PHD was 4.9 g.

COMPARISON OF PREDICTIVE SIMULATIONS AND FULL-SCALE CRASH TESTS
Sequential views from FE simulations and tests are presented in Figures 7 and 8 for the small passenger car TB 11 and HGV TB 71 impacts, respectively. Both simulation predictions show a good agreement with test outcomes in terms of impact behavior and post-impact vehicle trajectory.

The ASI predicted by the small car impact computer simulation was 1.43. Geo Metro test results showed an ASI of 1.69. The difference between the predicted value and the test outcome might be partially related to the need for modeling improvement of the small car FE model. FE simulation accuracy could be further improved by enhancing suspension modeling of the small passenger car to capture a more faithfully realistic interaction between the vehicle and the barrier.

A program called the Roadside Safety Verification and Validation Program (RSVVP) was developed for validation of numerical models in roadside safety (9). This program was used to compute the comparison metrics for a quantitative validation of the small passenger car FE impact model. In this case, RSVVP is applied to the FE simulation after the full-scale crash test is already conducted. Thus, the purpose is to evaluate whether the FE small car simulation could have been considered quantitatively predictive of the crash test outcomes.

This quantitative verification approach is based on the comparison of acceleration and angle rate curves from both simulation and test data according to Sprague and Geers (S&G) MPC and analysis of variance (ANOVA) metrics. The acceleration and angle rate histories of the vehicle are collected in LS-DYNA using a rigid brick element defined by the card *ELEMENT_SEATBELT_ACCELEROMETER and rigidly linked to the vehicle at its center of gravity (J). Before computation of the metrics with the RSVVP program, each curve was filtered and synchronized by minimizing the absolute area of the residuals.
SUMMARY AND CONCLUSIONS

This paper shows a successful design analysis of a high-containment bridge rail required for a viaduct by the Hong Kong Special Administrative Region. Computer modeling was employed to define the various steps of the test article design. FE simulations were able to correctly predict the vehicle’s trajectory, interaction with the barrier, and test article post-impact damage that resulted in the full-scale crash tests.

Both tests with the small passenger car and HGV impacting the bridge rail are considered successful. The Hong Kong bridge rail meets the requirements of EN 1317 (2,3). The profile of the concrete parapet provided stability for small passenger cars, and the strength of the rail was adequate for containment of HGVs weighing up to 30,000 kg.
REFERENCES


A New MASH-Compliant Guardrail System for Placement on Slopes

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ABSTRACT

The 2002 American Association of State Highway and Transportation Officials Roadside Design Guide recommends that a W-beam guardrail be installed with its back positioned 0.61 m (2 ft) in front of a slope break. In rolling and mountainous terrain or in locations with restrictive environmental conditions, it is often difficult to provide this distance. As a result, designers must evaluate the trade-offs between a reduced shoulder width and guardrail installations that are less than optimal. In the most restrictive site conditions, the post lengths are excessively long, which complicates system installation and maintenance repairs. The project focused on designs that minimized post length. A 31-inch high W-beam guardrail system with 8-ft-long posts, spaced 6 ft 3 inches on centers, met Manual for Assessing Safety Hardware (MASH) test level 3 criteria when placed with the face of the rail over the slope break point on a 2H:1V fill slope.

INTRODUCTION

In rolling or mountainous terrain or in areas with restrictive environmental conditions, it is often difficult to provide sufficient width behind the guardrail posts for a typical W-beam guardrail (designated as SGR04a). The 2002 American Association of State Highway and Transportation Officials (AASHTO) Roadside Design Guide (1) specifies a minimum desirable dimension of 2 ft from the back of the post to the slope break point on a fill slope. This width behind the post offers soil resistance to post rotation that is similar to the conditions on flat terrain. This approach provides consistency in guardrail performance.

In rolling or mountainous terrain, providing additional width for the roadway cross section often results in greater fill heights, which may extend beyond the available right of way. In many environmentally sensitive areas, minimizing encroachment into the sensitive area is a strategy used in mitigating environmental impacts. Many states have approached this issue by extending guardrail post length and/or decreasing the post spacing, along with reducing the dimension behind the guardrail post. Some of these designs have not been fully crash tested. In some instances, states have used posts as long as 11 ft, which makes post installation and maintenance repairs more difficult.

This research project was to use standard W-beam guardrail components in the development of a W-beam guardrail system suitable for placement at the top of a 2H:1V fill slope. While it was recognized that longer posts would be necessary, the research focused on minimizing post length to the greatest extent possible. Initial development efforts targeted a W-beam guardrail system with a 27-inch mounting height to the top of the rail. The 27-inch high system was evaluated through bogie testing, computer simulation, and full-scale crash testing under National Cooperative Highway Research Program...
(NCHRP) Report 350 (2) criteria. During the full-scale crash test, the pickup overturned after having been restrained and redirected by the guardrail system. Post-crash analysis explored other design options, with the project sponsors ultimately electing to pursue this design using a 31-inch high W-beam guardrail system to increase vehicle stability. The timing of this redesign coincided with the adoption of Manual for Assessing Safety Hardware (MASH) (3) criteria, and the sponsors elected to pursue the project in accordance with MASH guidance.

BACKGROUND

The recommended AASHTO placement of 2 feet provides sufficient width behind the post for the soil to resist excessive post rotation. Posts typically begin to rotate through the soil until the vehicle is restrained or the soil resistance becomes so great that the post yields and the energy is transferred to the adjacent posts. If the post is placed too close to the slope break point on steep fill slopes, the cohesive soil strength may be insufficient to restrict post rotation. In rolling and mountainous terrain conditions or in locations with restrictive environmental conditions, it is difficult to provide this distance. As a result, designers must evaluate the trade-offs between a reduced shoulder width and guardrail installations with longer posts to provide adequate soil embedment. Some states have used designs with posts up to 11 feet long. These excessively long posts complicate system installation and maintenance repairs. Some designs currently in use may not have been fully crash tested.

ENSCO, Inc. conducted the earliest known research with guardrail placement on slopes in 1993. This testing included pendulum tests on single posts and followed by three full-scale crash tests. Two tests were considered successful under 1981 NCHRP Report 230 criteria, with a large sedan impacting the G4(1S) guardrail system installed at the break point of a 2H:1V slope. One of the systems used a configuration with 6 foot long posts, and the other configuration used 7 foot long posts. The installation with 7 foot posts had a smaller deflection and less vehicle impact speed change than the installation with 6 foot long posts.

The Midwest Roadside Safety Facility (MwRSF) in 2000 (4) performed a battery of bogie tests and a test level 3 crash test of a 27 ¾ inch high guardrail system with 7 foot long W152×13.4 (W6×9) steel posts placed 37.5 inches on centers. This system was successfully crash tested under NCHRP Report 350 criteria with the posts placed at the break point on a 2H:1V fill slope.

Subsequently in 2006, researchers at MwRSF performed a battery of bogie tests with steel posts ranging from 6 feet to 9 foot long, placed at the break point on a 2H:1V fill slope, along with two full-scale crash tests. The 27.75-inch high rail design was unsuccessful according to MASH test evaluation criteria. However, the 31-inch high guardrail system using the 9 foot long W6×9 steel posts placed 6 feet 3 inches on centers successfully passed MASH test 3-11 criteria.

In 2008, the Texas A&M Transportation Institute (TTI) performed a series of bogie tests with steel posts ranging from 6 feet to 9 feet long as well (5). Some were placed on a 2H:1V fill slope, and some were placed on a 1.5H:1V fill slope. All were placed 1 foot down from the slope break. Based on these tests and subsequent simulations, a guardrail on a slope system was recommended for full-scale crash testing. The system was a 27-inch high guardrail system with 8 foot long W6×9 steel posts placed 37.5 inches on centers. This system was crash tested under NCHRP Report 350 TL-3-11 conditions with the posts placed at the break point on a 2H:1V fill slope. The test was not successful according to NCHRP Report 350 test evaluation criteria.

RESEARCH APPROACH

The research team focused on system development under MASH conditions since AASHTO adopted MASH as crash-testing guidelines in 2009. The research team developed a detailed finite element model of the new design to investigate design performance under MASH test conditions. Also, the new Silverado vehicle model developed by the National Crash Analysis Center (NCAC) was used to simulate the MASH 2270P test vehicle (6).

The post is comprised of different thicknesses to accurately represent the shape of a W6×9 steel post. Researchers used 18,240 shell elements for modeling the posts. Additionally, the W-beam model contains a more refined element mesh than the previously used W-beam models, so it can capture deformation more realistically. Researchers used 182,304 shell elements for modeling the W-beam segments. Both the post and the W-beam models are shown in Figure 2. The end terminals and the remaining portion of the length of need (LON) rail are represented by spring elements that are connected to each end of the modeled W-beam. These springs elements have a combined stiffness representative of typical end terminals.
The first design change was to increase the rail height from 27 inches to 31 inches relative to the flat terrain. The cross-section views of the new system design and the model are shown in Figure 3.

Initial simulation was conducted to capture the steady-state condition. Basically, the model was run without the vehicle to account for the gravitational load until stresses in the soil are in equilibrium, as shown in Figure 4. Once that state is achieved, impact simulations can be conducted.

Simulation Results
In the first simulation case, the 8-ft-long W6×9 steel posts were placed 3 ft 1.5 inches apart with standard 8-inch wood blockouts. The front wheel of the truck snagged on the posts of the guardrail system during impact, as shown in Figure 5. This snag was a cause of concern for half-spaced placement of posts.

In the second simulation, the 8 foot long W6×9 steel posts were spaced 6 ft 3 inches apart with standard 8 inch wood blockouts. The steel posts were placed off-splice, meaning that no steel post was connected to any splice. The model for this case is shown in Figure 6.

This system was successful in containing and redirecting the 2270P vehicle according to MASH evaluation criteria. The rail system reached a maximum deflection of around 3.44 feet at 0.22 s. The deflection value is considered reasonable for typical strong post guardrail systems on flat terrain. Figure 7 shows front left wheel interaction with bent posts and the wood blockout.

The vehicle model used for simulation is the Chevrolet Silverado model, which was developed by NCAC. This

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The maximum predicted roll of the Silverado model occurred at 0.2941 s, with a value of −11.7 degrees. The maximum predicted pitch of the Silverado model occurred at 0.6239 s, with a value of −12.4 degrees. The maximum predicted yaw of the Silverado model was 31.1 degrees and occurred at 0.4210 s.

**Full-Scale Crash Tests**

The guardrail on the slope system consists of a 181 feet 3 inch total length of a 12 gauge W-beam mounted on W6×8.5 steel posts. The guardrail system was comprised of a 106 feet 3 inch length of need section and a 37.5 feet long terminal on each end. A 2H:1V sloped ditch was excavated behind the rail to represent the sloped terrain. The ditch was centered along the installation length and was 75 foot long and 8 feet wide (to the bottom of the ditch).

Along the sloped section, the 8 foot long posts were placed at a spacing of 6 feet 3 inches. These were post 10 through post 21, as shown in Figure 8. Standard size blocks (8 inches × 6 inches × 14 inches) were used in the length of need section. The splices were placed off posts and blockouts. A downstream view of the system installation is shown in Figure 9.

**TEST DESCRIPTION**

The 2006 Dodge Ram 1500 pickup truck, traveling at an impact speed of 63.9 mph, impacted the guardrail on slope 11.1 inches upstream of post 14 at an impact angle of 25.0 degrees. Shortly after impact, at 0.010 s, post 14 began to deflect toward the field side, and at 0.012 s, post 15 began to deflect toward the field side. Post 13 began to deflect toward the field side at 0.024 s, and the vehicle began to redirect at 0.041 s. At 0.056 s, post 12 began to deflect toward the field side, and at 0.141 s, the left front tire continued to ride under the rail and contacted post 15. The rear of the vehicle contacted the guardrail at 0.179 s, and the left front tire rode under the rail and contacted post 16 at 0.233 s. At 0.263 s, the vehicle began to travel parallel with the guardrail traveling at a speed of 51.1 mi/h. The left front tire, still under the rail element, contacted posts 17 and 18 at 0.333 s and 0.433 s, respectively. At 0.549 s, the vehicle lost contact with the guardrail and was out of view of the overhead camera, and exit speed and angle were not obtainable. Judging from vehicle tire path, the exit angle was estimated to be 10 degrees. Sequential photographs of the test period are shown in Figure 10.
FIGURE 10: Sequential Photographs for Test 405160-20-1 (Overhead and Frontal Views)

TEST DESCRIPTION
The 2006 Kia Rio, traveling at an impact speed of 60.3 mph, impacted the guardrail on slope 36.0 inches upstream of post 15 at an impact angle of 25.9 degrees. Shortly after impact, posts 14 and 15 began to deflect toward the field side of the guardrail, and at 0.036 s, the left front tire contacted post 15. The vehicle began to redirect at 0.050 s, and post 16 began to deflect toward the field side at 0.062 s. At 0.089 s, the left front tire contacted post 16, and at 0.105 s, post 17 began to deflect toward the field side. Post 18 began to deflect toward the field side at 0.111 s, and the left front tire contacted post 17 at 0.179 s. At 0.275 s, the vehicle began to travel parallel with the guardrail at a speed of 37.5 mph. The left front tire contacted post 18 at 0.276 s. At 0.545 s, the vehicle lost contact with the guardrail and was traveling at an exit speed and angle of 31.3 mph and 32.3 degrees, respectively. The test summary sheet is shown in Table 2.

RESULTS AND CONCLUSIONS
A new guardrail on slope system was developed and tested successfully per MASH TL-3 conditions. Finite element simulation using LS-DYNA was used to assess the system performance prior to testing. The guardrail system passed the evaluation criteria for both 3-11 and 3-20 tests. The system uses shorter posts, standard blockouts, and standard post spacing. Moreover, the system can be placed 1 foot down from the slope break. These advantages give roadway designers more design space when planning mountainous roadway.
TABLE 2: Test 405160-20-2 Summary Sheet

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<th>General Information</th>
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<tr>
<td>Guardrail</td>
<td>Speed</td>
<td>7 degrees</td>
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<tr>
<td>Name</td>
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<td>Vehicle Snagging</td>
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<td>WSDOT guardrail on slop</td>
<td>Angle</td>
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<tr>
<td>Installation Length</td>
<td>32.3 degrees</td>
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<td>181 ft 3 inches</td>
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<td>Material or Key Elements</td>
<td>Impact Velocity</td>
<td>Dynamic</td>
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<td>12 gauge W-beam mounted on W6x8.5 steel posts on 2H:1V slope</td>
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<td>2.7 ft</td>
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<td>Soil Type and Condition</td>
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<td>164 lb</td>
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REFERENCES


**Chairman (2013-2015)**  
Abdullah A. Al-Mogbel  
Municipality of Riyadh

**Vice Chairmen (2013-2015)**  
Jeffrey R. Reed  
Valley Slurry Seal Company  
T. Peter Ruane  
ARTBA  
Thomas Topolski  
The Louis Berger Group

**Treasurer (2013-2015)**  
Lester Yoshida  
Delcan Corporation

**President & CEO and Secretary**  
C. Patrick Sankey  
International Road Federation

**Elected Directors to Serve on Executive Committee (2014-2015)**

<table>
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<tr>
<th>Elected Directors</th>
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<tr>
<td>Jacobo Diaz</td>
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<tr>
<td>Transpo Industries</td>
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**IREF Chairman (2014-2015)**  
Essam Radwan  
University of Central Florida

**Past Chairman**  
Brian J. Stearman  
Delcan Corporation

**Directors (2013 – 2015)**

<table>
<thead>
<tr>
<th>Directors</th>
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<tr>
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<tr>
<td>Arthur Dinitz</td>
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<tr>
<td>* Denotes Ex-Officio Member</td>
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**Directors (2014 – 2016)**

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The INTERNATIONAL ROAD FEDERATION is a full-service membership organization founded in Washington, DC in 1948. The IRF is a non-governmental, not-for-profit organization with the mission to encourage and promote development and maintenance of better, safer and more sustainable roads and road networks around the world. Working together with its members and associates, the IRF promotes social and economic benefits that flow from well-planned and environmentally sound road transport networks and advocates for technological solutions and management practices that provide maximum economic and social returns from national road investments.