A REVIEW TO DETERMINE THE NEED, LOCATION, AND DESIGN OF RUNAWAY LANES IN BRITISH COLUMBIA

by

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Abstract

British Columbia's steep mountain highways require runaway lanes to assist heavy vehicles if they lose control. Runaway lanes, or speed control lanes have been implemented in Canadian and American highways for over 40 years. They are used in mountainous regions as a primary tool to minimize the risk and consequences of heavy vehicle runaway accidents. Insufficient vehicle and brake maintenance, improper use of brake check facilities prior to descending steep grades, excessive descent speeds and/or relying primarily on the service brakes to control speed on downgrades leading to excessive brake heating results in a loss of brake capacity. Also, the lack of knowledge of the highway terrain to be encountered contributes to a condition where the driver has insufficient brake capacity to reduce vehicle speed for the road terrain being encountered. Runaway lanes are therefore placed at various locations along steep and/or lengthy downgrades where drivers can utilize them soon after realizing that a potential runaway situation exists.

A comprehensive review of previous work relating to runaway lane design was conducted. Full scale tests of several runaway lane designs currently existing in British Columbia were carried out to identify any design deficiencies. In addition, a brake temperature model was evaluated and modified to reflect the eight axle truck configurations existing in British Columbia to devise a simple runaway lane warrant.

The final outcome of the research concluded that the arrestor bed type runaway lane is the most suitable for safely stopping the different truck configurations currently existing in British Columbia. In addition, a simple runaway lane warrant based on brake fade temperature was devised and calibrated for an eight axle B-train truck configuration.

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1.0 Introduction

British Columbia's steep mountain highways require runaway lanes to assist heavy vehicles that have lost control. It should be emphasized that truck runaway lanes, or speed control lanes are a last resort device for trucks that have lost braking capacity on steep grades, and are used in extreme circumstances.

There are several factors that contribute to trucks losing control on steep grades. Insufficient scheduled and/or daily vehicle and brake maintenance, improper use of brake check facilities prior to steep grades, excessive descent speeds and/or the use of service brakes to control speed on downgrades leading to excessive brake heating and resulting loss of brake capacity, and lack of driver knowledge and experience of highway terrain to be encountered. Out of province drivers seem to account for the majority of runaway accidents in British Columbia due to their inexperience in driving steep mountainous terrain.

1.1 Research Objectives

The British Columbia Ministry of Transportation and Highways currently does not have design or warrants for runaway lanes. Furthermore, there are no definitive standards or warranting methods on runaway lanes documented in Canada. The increasing reliance on commercial trucks in the movement of goods, necessitates the creation of such a warranting process. Current placement of runaway facilities are often incorrect and the facilities may be inadequate to accommodate many commercial vehicles. This inadequacy results in truck drivers taking unnecessary risk by either avoiding the runaway facilities and attempting to run out the grade or by using a substandard facility at great risk to themselves, their equipment and cargo.

Most of the runaway lanes in British Columbia are based on designs used in the United States. However, these designs are based on truck configurations that are different than those currently existing in British Columbia. In British Columbia, trucks can have up to nine axles and can weigh up to 63,500 kg, as opposed to the United States where vehicles are limited to five axles and 36,000 kg.

In recent times, there has been an increased interest, from within the Ministry and other agencies, in the installation of runaway lanes at high risk locations. The tragic outcomes from catastrophic runaway accidents on British Columbia highways has emphasized the need for a strategy in locating, designing, signing, and maintaining runaway lanes at high risk locations.

1.2 Scope of the Project

Previous work by the British Columbia Ministry of Transportation and Highways, Highway Safety Branch was related to the warranting methods and design details for runaway lane facilities. The warranting methods, however, were based on the FHWA grade severity rating system. The FHWA system does not account for the different type of truck configurations in British Columbia.

The purpose of this research project is to review and enhance previous work completed by the Highway Safety Branch, and is broken down into the following components: 1) comprehensive literature review to identify current standards, 2) full scale testing of runaway lane designs currently existing in British Columbia to identify deficiencies and possible improvements, 3) documenting design standards for runaway lane installations for the British Columbian environment 4) evaluating and modifying the FHWA grade severity rating system for use in British Columbia, and 5) devising a simple warranting system for runaway lane installation based on brake temperature models, using an eight axle B-train as the design vehicle. It is hoped that the research presented here, along with better driver education programs, and stricter enforcement and stiffer penalties for poorly maintained vehicles, will contribute to reducing and possibly eliminating truck runaway accidents on British Columbian highways.

2.0 Literature Review

The literature review is broken down into four separate sections. In preparation for the full scale testing, documented studies pertaining to runaway lane experience were evaluated to gain background into testing procedures and to reveal possible difficulties to be encountered during such tests. The second section reviews current recommended guidelines for runaway lane design to determine whether modifications are necessary for the different type of truck configurations, and environmental conditions existing in British Columbia. The final two sections deal with grade severity rating systems, and warranting systems, and how they may be used to determine whether installation of runaway lanes are required.

2.1 Runaway Lane Experience and Testing

The earliest tests to evaluate the performance of arrestor beds were conducted by Jehu and Laker in 1969 [30]. The tests involved testing several different types of aggregate material including rounded gravel, angular gravel, and a lightweight aggregate known as 'LYTAG'. The authors concluded that mean values of deceleration obtained without application of a vehicle's brakes were dependent upon the characteristics of the arrestor bed including the size and shape of stone, but independent on the vehicle type and entry speed. Test results indicated that the rounded gravel and lightweight aggregate were able to stop a vehicle in less distance than compared to the angular gravel. Deceleration rates were in the range of between 0.22 g to 0.33 g for a passenger car, and 0.21 g to 0.24 g for a 15,500 kg articulated vehicle.

Laker continued the testing of the lightweight aggregate, 'LYTAG', in 1971, for its application as an arresting material in arrestor beds. From these tests, he derived an empirical formula to determine the length of bed required to stop a runaway vehicle. The equation is as follows:

$$L = \frac{v^2}{127}$$

where L= the length required in metres, v = the entry velocity in kph. This equation is based on an aggregate depth of 0.38 to 0.45 metres of 'LYTAG'.

Afferton, et al [2] conducted arrestor bed tests in 1976 using a local one cm diameter washed pea gravel. Eight different gravel bed configurations were tested, with the 460 mm pile bed being the most effective. The results indicate that this design is capable of stopping a standard size automobile impacting the bed at 89 kph in 31 metres. The authors cautioned that since only one type of vehicle weight class was tested (1,700 kg), the proposed configuration as an entrapment system for heavier vehicles would require verification through full-scale testing of representative vehicle types.

Allison et al [5] conducted tests of a truck-arrestor system that consisted of a 158 metre long bed of 0.6 metre deep screened gravel, backed by an array of 88 sand-filled plastic barrels in 1978. Three separate trials runs with a 16,650 kg dump truck at speed of 34, 66, and 90 kph demonstrated that the decelerations experienced were similar to those experienced in panic stops on dry pavement. The authors suggest that application of deicing chemicals appears to be necessary to prevent the freezing of the gravel bed in the winter.

In 1979, Young [52] evaluated the performance of an arrestor bed built on a descending grade (-5.5%). The 366 metre long, 7.9 metre wide, and 457 mm deep gravel arrestor bed incorporated an adjacent service lane to aid in the removal of entrapped vehicles. Results concluded that the arrestor bed was adequate for vehicles weighing up to 23,000 kg, and entering at speeds up to 130 kph.

Booze [11] evaluated potential methods for stopping heavy runaway vehicles on long, steep downgrades and included devices for arresting aircraft in emergency landing or take-off situations, crash cushions or barriers, retarders which are a part of the vehicle, and escape ramps utilizing an arresting material and/or an ascending grade. His study also examined the energy dissipation properties of a variety of materials covering a variation of particle size, particle shape, vehicle velocity, and wheel load. Testing indicated that rolling resistance increases with bed depth up to 457 mm, and that smooth, spherical particles exhibited much higher rolling resistance than irregular or angular particles.

The Mt. Vernon Canyon Runaway Truck Escape Ramp [26] was another example of an arrestor bed built on a descending grade. The runaway lane was completed in 1979 and to date, has stopped more than fifty-three runaway or potentially runaway trucks. Only two trucks sustained damage and there were no injuries or fatalities in the runaway lane.

In 1982, Brown [13] constructed two arrestor beds on a level airstrip in Australia to evaluate their performance on rigid body and articulated vehicles entering at speeds ranging from 60 to 90 kph. One bed utilize dune sand, while the other contained river gravel. Brown concluded that dune sand produced similar deceleration rates when compared to the river gravel, provided the sand was in a dry uncompacted state. He also concluded that as the number of axles and the vehicle load increased, the lower the deceleration rates became.

In an attempt to find a suitable aggregate gradation resistant to contamination and freezing conditions Derakhshandeh [17] monitored several truck runaway lanes in Colorado over a two year period. The study concludes with some guidelines for selecting the most appropriate aggregate type for arrestor beds located in cold climates.

In 1988, Wambold et al [48] derived an empirical equation to predict stopping distances based on tests on existing arrestor beds in Pennsylvania. The study concluded that smooth, rounded, uncrushed gravel of approximately a single size was the most effective arrestor bed material. The best size appeared to be near 0.5 inch in diameter. The tests also revealed that a bed with at least 36 inches of gravel gives the same results as a bed as deep as 8 feet. A bed depth of at least 42 inches was recommended.

In 1991, the Arizona Department of Transportation reviewed the 'DRAGNET' vehicle arresting system for application in truck runaway lanes. The results of the evaluation showed that a 'DRAGNET' system was more expensive to install when compared to an arrestor bed. In addition, in order to keep maximum ridedown deceleration rates below 0.45 g, and incorporate a factor of safety of two, the resulting stopping distance needed would be greater than a comparable arrestor bed. Since the evaluation was purely analytical, and no full scale testing was performed, the study concluded that the 'DRAGNET' system was not suitable as a stand alone system for application in truck runaway lanes, until further testing was performed.

In 1992, the Arizona Department of Transportation conducted 102 full scale entries distributed among four of Arizona's seven arrestor beds. At the highest design entry speed, in beds of comparable aggregate and slope, an average deceleration rate of 0.41 g was achieved in a bed that had an average depth of 39.2 inches, and an average deceleration rate of 0.34 was achieved in a bed that had an average depth of 12.8 inches over the average stopping distance.

In mid 1994, Main Roads in Western Australia [32] became the first to document tests of vehicles weighing more than 36,000 kg and having more than 5 axles entering arrestor beds at high speeds without damage. The study concluded that arrestor beds were suitable for safely stopping vehicles weighing up to 78,800 kg and having up to 9 axles.

2.2 Recommended Guidelines for the Design of Runaway Lanes

In 1978, Williams [50] published a report presenting the state of the art practice synopsis of runaway lanes and an overview of existing facilities in regard to design, construction, and practical operational techniques. The report provides a detailed description of the runaway lanes known to be existing in the United States at the time of publication.

Eck [20] performed a similar report several years later, and documented the United States' practice and experience in the use and location of truck escape ramp facilities. Eck forwarded a questionnaire to state highway agencies to determine the key factors used when determining the need for runaway lanes. According to the survey the following factors, ordered in decreasing importance, are as follows: runaway truck accident rate, length of grade, percent grade, percent trucks, condition at bottom of grade, average daily traffic, horizontal curvature, accident severity, available right of way, and topography. The questionnaire concluded that accident history and experience was cited as the number one contributing factor in determining the need for a runaway lane facility. When questioned about where to locate the runaway lane, the state highway agencies cited the following factors, ordered in decreasing importance: earthwork costs, horizontal alignment, accident location, condition at bottom of grade, availability of right of way, truck driver input, speed of out of control vehicle, and length of grade. The questionnaire concluded that existing runaway lanes were installed mainly by engineering judgment, and the location was dictated by minimizing the cost of earthwork.

Ballard [9] also conducted a state wide survey of existing runaway lanes in 1983 and concluded that there were 6 basic types of facilities including: sandpile, gravity ramp, ascending grade arrestor bed, horizontal grade arrestor bed, descending grade arrestor bed, and roadside arrestor bed. Similar to previous studies, the report did not offer formal guidelines for design of runaway lanes, but only a summary of existing designs.

Documented formal guidelines for design of runaway lanes include the Colorado Design Manual for Truck Escape Ramps [26], ITE Truck Escape Ramps: Recommended Practice [29], AASHTO's Policy on Geometric Design of Highways and Streets [6], and NCHRP Synthesis on Truck Escape Ramps [51]. The province of British Columbia's Ministry of Transportation and Highways, currently does not have a formal guideline for design of runaway lanes. Furthermore, the Transportation Association of Canada (TAC) only makes a brief reference to runaway lanes, and again, does not have formal guidelines for runaway lane design.

2.3 Grade Severity Rating Systems

Grade severity rating systems (GSRS) existed as early as the 1960's and were based solely on length and grade characteristics. During the late 1970's, the Federal Highway Administration conducted several studies to come up with a grade severity rating system that not only included topographical conditions, but included vehicle parameters as well. The FHWA grade severity rating system predicted the temperature of the brakes of a vehicle descending a grade, and based on these temperatures, recommended safe descent speeds. The GSRS was calibrated, however, for trucks weighing less than 36,000 kg and having less than or equal to five braking axles. The University of Michigan (UMTRI), later developed a brake temperature model similar to the one used in the FHWA GSRS, but the brake temperature model was not limited to 5-axle trucks. This model, therefore, is more appropriate for use in British Columbia, where truck configurations can have up to 9 axles. Both the FHWA GSRS and UMTRI's brake temperature model are useful in determining suitable locations for installation of runaway lanes, by predicting where brake fade will most likely occur due to the grade and length of the downgrade.

2.4 Warranting Systems for Runaway Lanes

To date, there are no formal guidelines for warranting of runway lanes in Canada. The FHWA GSRS can be considered a type of warranting system, but it is calibrated for truck configurations in the United States, and will need some modification if it is to be used in British Columbia. The most thorough research to date on the topic of runaway lane warrants is by Eck [19]. His research consisted of six phases and included: a mail questionnaire to determine highway agencies' experience and practices relative to runaway lanes, preparation of the state of the art experience bibliography on runaway lanes, a second questionnaire to determine truck drivers' perceptions of the runaway vehicle problem, collection and analysis of runaway vehicle accident data, formulation of warrants for the use of runaway lanes, and formulation of a methodology for determining appropriate locations for runaway lanes. Again, the warrants were based on experience in the United States, and due to the differing truck configurations, and environment, cannot be readily applied in British Columbia.

3.0 Full Scale Testing of Runaway Lanes in British Columbia

3.1 Introduction

The objective of the test was to observe and record the performance of a fully loaded, five axle, single articulation tractor-trailer, as it negotiated different types of truck runaway lanes. Two types of truck runaway lanes were investigated: the descending grade arrestor bed, and the ascension lane. The information of interest for trucks entering a runway lane are: 1) entry speed, 2) deceleration in the runaway lane, 3) vehicle stability in the runaway lane, and 4) the distance required for the vehicle to come to a rest. The data obtained from the planned experiments addressed the above issues

3.2 Test Conditions

3.2.1 Test Facilities

The full scale testing of a fully loaded, five axle, single articulation tractor-trailer was performed at two locations. The ascension lane test was performed at Pennask Hill, on Route 97C, west of Peachland. The arrestor bed test was performed at Hamilton Hill, on Route 97C, east of Merritt.

Ascension Lane

The ascension lane at Pennask Hill consists of a steep slope, and relies primarily on gravity to decelerate a vehicle and bring it to a rest (see Figure 3.1). The lane maintains a constant ascending grade of 25 % and has a total length of approximately 185 m. The average width of the lane is 7 m. As shown in Figure 3.2, entrance to the ascension lane is in total view of the driver and signs clearly mark the entrance. The paved asphalt leading into the ascension lane abruptly drops off and becomes a mixture of hard packed dirt and fine sediment (see Figure 3.3). This material constitute the surface conditions for the ascension lane (see Figure 3.4) . No small aggregate bed, or a similar type of facility was present near the top of the lane to keep the vehicle in place once it had stopped. A ravine on the downslope side of the ascension lane acts as erosion control and prevents sediment from flowing onto the highway.

Figure 3.1 Ascension Lane Profile at Pennask Hill

Figure 3.2 Ascension Lane at Pennask Hill

Figure 3.3 Entrance to the Ascension Lane at Pennask Hill

Figure 3.4 Top View of the Ascension Lane and Surface Conditions

Descending Grade Arrestor Bed

The descending grade arrestor bed at Hamilton Hill can be broken down into three different segments: 1) the entrance lane prior to the arrestor bed, 2) the arrestor bed, and 3) the service lane adjacent to the arrestor bed. The entrance lane prior to the arrestor bed has a total length of approximately 185 m and is composed of a thin layer of hard packed angular gravel (approximately 19 mm in diameter). The arrestor bed immediately follows this section and consists of a bed of 19 mm diameter pea gravel, as shown in Figure 3.6. This bed of gravel serves to decelerate vehicles as a result of rolling resistance provided by the aggregate. The arrestor bed has a descending grade of approximately 4 % and has a total length of 410 m. The average width of the bed is 5.5 metres. The depth of pea gravel is 50 mm at the entrance, and gradually reaches a maximum depth of 450 mm at a distance of 10 m away. The depth the of pea gravel then remains at a constant 450 mm for the remainder of the bed.

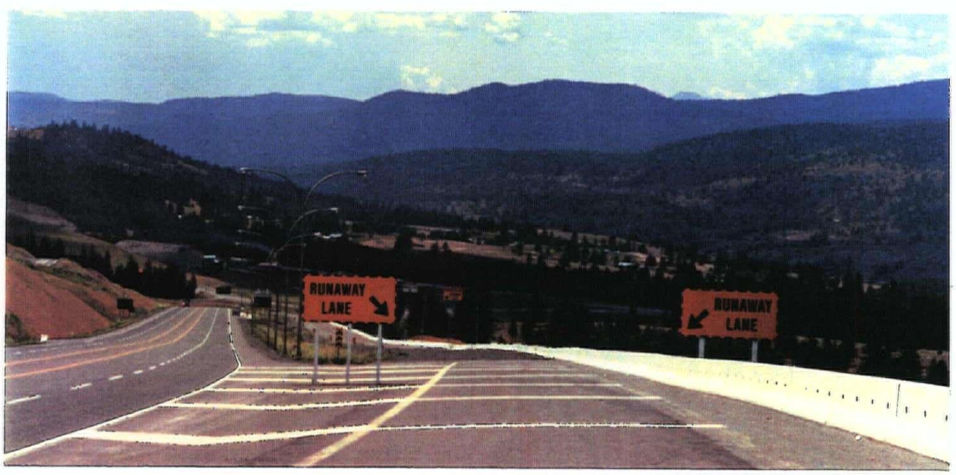

Figure 3.5 Entrance to the Runaway Lane at Hamilton Hill

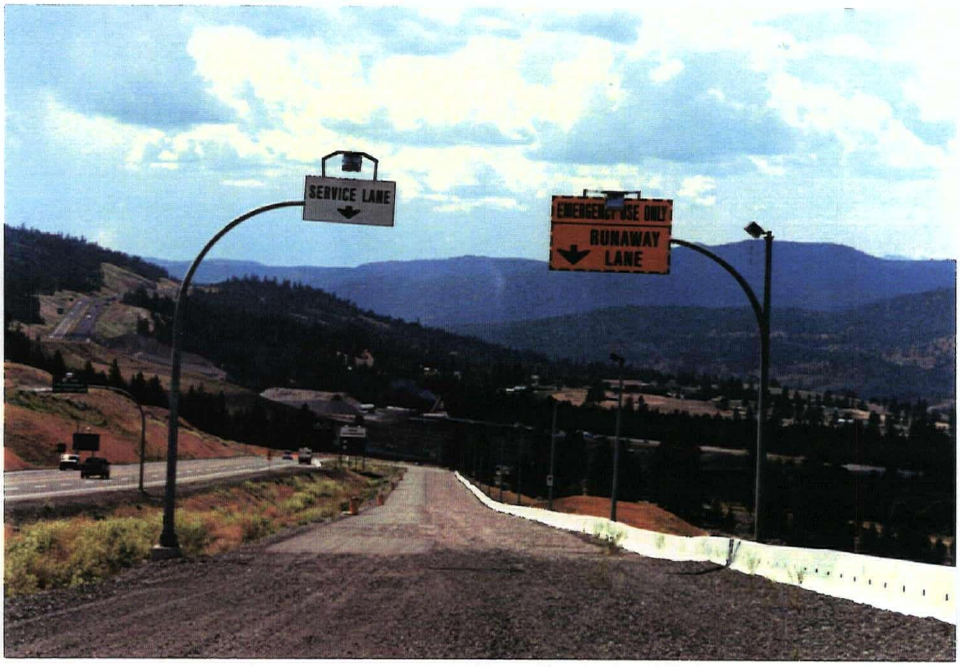

Figure 3.6 Gravel Entrance Lane Prior to the Arrestor Bed, the Arrestor Bed, and the Service Lane

Figure 3.7 End Conditions of the Arrestor Bed

A 3.6 m wide service lane runs adjacent to the bed, and provides access to maintenance vehicles. Hold down anchors spaced 90 m apart run along the edge of the service lane and aid in the removal of vehicles trapped in the bed. Luminaires, installed along the edge of the arrestor bed, illuminate the facility during hours of darkness.

3.2.2 Test Vehicle

The test vehicle chosen was a five axle, single articulation tractor-trailer. The tractor was supplied by Peterbilt Trucks Pacific Inc.. For further driver protection, an aluminum bulkhead was fitted behind the cab. The trailer, as supplied by DCT Chambers Trucking Ltd., is a typical flatbed style with a deck surface consisting of rough wooden planks bolted to the frame. The trailer was loaded with 8 banded units of economy grade lumber in the configuration shown in Figure 3.8. The load was secured using webbing that was ratcheted down onto the trailer (approximately 2-3 per banded unit). There was concern, however, that individual pieces of lumber in the top bundles would shift forward under high deceleration rates, thus providing a potential safety hazard for the driver or damage to the tractor cab. It should be noted that these top bundles were not butted against metal stakes, like the bottom bundles (see Figure 3.9). To solve this problem, sheets of plywood were strapped, using metal bands, to the front and back end of the top bundle located closest to the cab (see Figure 3.10). No modification was made to the top rear bundle because it was less likely to cause damage to the cab, and was left unsecured to observe whether this was indeed a problem. Metal stakes, reinforced by chains at the front end of the trailer, and joined by a piece of plywood, prevented the bottom loads from shifting into the cab.

The vehicle weights, as determined using a local truck scale, were measured as follows:

Table 3.1 Test Vehicle Weights

	Steering Axle	Drive Axle	Trailer
Unloaded	4630 kg	5580 kg	4290 kg
Loaded	4990 kg	15310 kg	17600 kg

Tape measures were used to document the physical dimensions of the vehicle.

Figure 3.8 Load Configuration on the Trailer

Figure 3.9 Front Stakes to Prevent Load Shift into the Cab

Figure 3.10 Plywood Strapped to the Top Bundle Closest to the Cab

3.2.3 Instrumentation and Data Aquisistion

To evaluate the performance of both the arrestor bed and the ascension lane, the vehicle dynamics was recorded. The different systems employed to document this testing were:

Electronic: G-Analysts, police radar system

Visual: S-VHS and Hi-8 video cameras, still photographs

Electronic Equipment

A total of four strategically placed G-Analysts (see Figure 3.11) were employed in the arrestor bed and ascension lane test: three mounted on the undercarriage of the trailer, and one mounted to the cab floor. Each G-Analyst unit consists of three accelerometers in orthogonal alignment and measure the lateral and longitudinal accelerations. The on-board recording system sampled data at a rate of ten samples per second, with a maximum capacity of 4800 samples (8 minutes of data). Data stored in each G-Analyst was immediately transferred to a portable computer after each test. Vehicle entry speeds were measured using a police radar handset.

The four G-Analysts were connected to a remote control unit which permitted 'flags' to be inserted in the recorded data, thus allowing the data to be synchronized. A twelve volt battery, mounted on the undercarriage of the trailer, provided the power source for the G-Analysts.

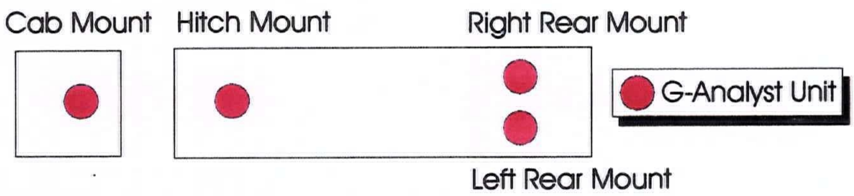

Figure 3.11 Mounting Locations of the G-Analysts

Visual Recording Equipment

Thorough documentation of this testing began with a complete photographic record of each test facility, instrumentation equipment, locations and the test vehicle. Still photographs were taken, along with pre and post-test documentation. Two S-VHS cameras (30 frames per second) and a Hi-8 camera (30 frames per second) were positioned as shown in Figure 3.12 during the arrestor bed test. The Hi-8 camera was placed next to the arrestor bed to capture tire and gravel interaction. Video reference markers were placed on the site such that photogrammetric techniques could be applied to these images to allow gross vehicle motions to be determined. The ascension lane test utilized the Hi-8 camera only, to capture qualitative information only.

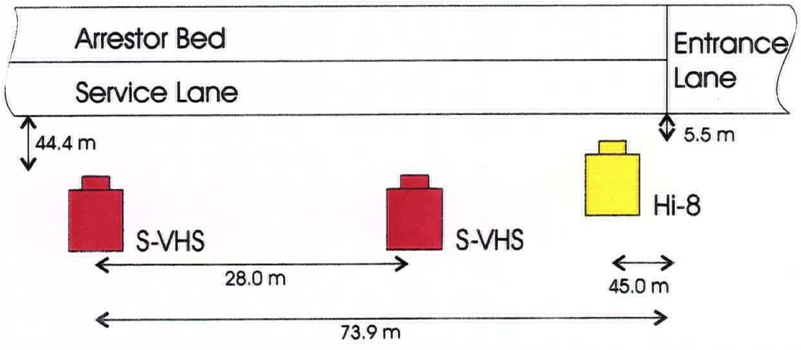

Figure 3.12 Placement of Video Cameras at the Arrestor Bed Site

3.2.4 Test Procedure - Ascension Lane and Arrestor Bed Testing

Pre-Test

The test vehicle and load were prepared as described in the test vehicle section. For each test, the vehicle was positioned at the top of the hill. The roadways were cleared of all traffic at this point. The video equipment and G-Analysts, used to record the tests, were started. The police radar gun and operator were positioned near the runaway lane entrance to measure entry speed.

Test

When the test-coordinator was satisfied that all traffic was cleared from the roadways and it was safe to proceed, the test vehicle began its descent. As the truck entered the runaway lane, 'flags' were inserted into the G-Analyst data via remote control, to synchronize the entry time.

Post-Test

After each test, the vehicle was measured and rephotographed to document damage, if any, that may have incurred during the test. Load shift was also checked by means of reference chalk lines drawn down the load prior to the test (see Figure 3.13).

Figure 3.13 Chalk Lines to Identify Load Shift

3.3 Test Results

A total of six individual tests were performed using the loaded tractor-trailer test vehicle: two braking tests on asphalt to simulate high deceleration rates, three tests up the ascension lane, and one test into the arrestor bed.

Braking Test

The braking tests on asphalt were performed using the loaded tractor-trailer test vehicle and served several purposes: 1) they allowed calibration of instruments on a level surface, 2) they simulated high deceleration rates that may be experienced on the actual runaway lanes, and 3) they permitted checking for load shift and possible instability. The braking test served primarily as a 'dry run', so that if any deficiencies were noted, they could be corrected before the actual tests into the runaway lanes were performed. Each test involved bringing the vehicle up to a constant speed of 40 km/hr and then applying the brakes firmly to bring the vehicle to a complete stop. A total of two runs were executed.

During the first run, the bottom loads shifted forward approximately 25-30 mm. Enough force was generated to cause the front stakes to bend slightly forward. The metal strapping and webbing on the loads, which were initially vertical, leaned slightly forward after the test. During the second run, no additional load shift was evident. The first run had closed the gap between the load and the front stakes, thus preventing the load from shifting any further forward. The vehicle stopped in a straight line and was stable laterally. Table 3.2 summarizes the maximum deceleration experienced due to hard braking on asphalt at 40 km/hr. Maximum decelerations were in the range of 0.50 g and were nearly consistent throughout instrumented locations on the test vehicle.

Table 3.2 Maximum Deceleration During Braking on Asphalt at 40 km/hr

	Cab Mount	Right Rear Mount	Left Rear Mount
Maximum Deceleration [g]	0.51	0.48	0.50

Figure 3.14 illustrates a typical deceleration profile from the braking tests:

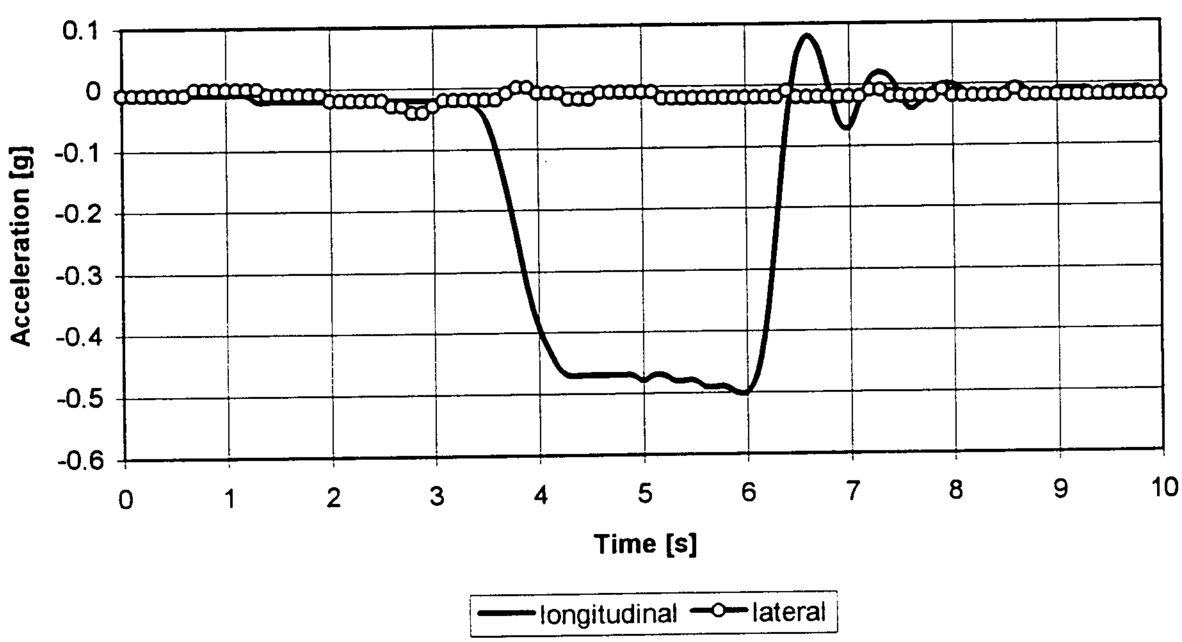

Figure 3.14 A Typical Deceleration Profile from the Braking Tests

At time t = 0 seconds, the test vehicle is traveling at a constant speed of 40 km/hr in a straight line; this is indicated by zero acceleration in both the longitudinal and lateral directions. At t = 3 seconds the brakes are applied firmly and the deceleration rapidly increases to its peak value and remains constant until the vehicle comes to a complete stop. As the vehicle decelerates, it tends to dive forward. Once it comes to a complete stop, the vehicle wants to bounce back to its original level position, and momentarily tilts backwards due to elastic rebound and inertia effects. It is this effect which causes the G-Analyst to detect a positive acceleration at t = 6.5 seconds. The deceleration profiles obtained from different G-Analyst locations for the braking tests were consistent and did not vary significantly in shape.

Ascension Lane Test

A total of three runs were performed up the ascension lane. During the first two runs, the brakes were applied at a pressure of approximately 5 psi, while ascending, to activate the rear brake lights. This served as a visual effect for the professional film crew on site documenting the tests. The brakes were not applied during the last run. The vehicle entered the ascension lane smoothly, with no sudden deceleration at the entrance. It was stable laterally and stopped in a straight line. A cloud of dust trailed the vehicle but quickly dispersed. After the vehicle came to a complete stop, it proceeded to slowly back down the hill without jack-knifing. After the three tests were performed, it was noted that the load had actually shifted slightly rearwards. Tension forces which existed in the webbing from the braking tests caused the load to be pulled back as the test vehicle dropped off the paved asphalt onto the ascension lane. At this point, the load experienced a vertical acceleration which allowed the strap tension to pull the load back slightly. No visible structural damage was incurred during the test. Table 3.3 summarizes the maximum decelerations experienced and the distances required to stop the vehicle for each test. Maximum decelerations were in the range of 0.30 g and were consistent throughout the instrumented locations on the test vehicle, and throughout each of the three tests. Figure 3.15 illustrates a typical deceleration profile from the ascension lane tests.

Table 3.3a Maximum Deceleration and Stopping Distances in the Ascension Lane

Ascension Lane Test # 1	
Entry Speed [km/hr]	70
Location of G-Analyst	Cab
Maximum Deceleration [g]	0.31
Stopping Distance [m]	130

Table 3.3b Maximum Deceleration and Stopping Distances in the Ascension Lane

Ascension Lane Test # 2			
Entry Speed [km/hr]	70		
Location of G-Analyst	Cab	Right Rear	Left Rear
Maximum Deceleration [g]	0.31	0.30	0.29
Stopping Distance [m]	138		

Table 3.3c Maximum Deceleration and Stopping Distances in the Ascension Lane

Ascension Lane Test # 3				
Entry Speed [km/hr]	Not Available			
Location of G-Analyst	Cab	Hitch	Right Rear	Left Rear
Maximum Deceleration [g]	0.34	0.34	0.33	0.34
Stopping Distance [m]	148			

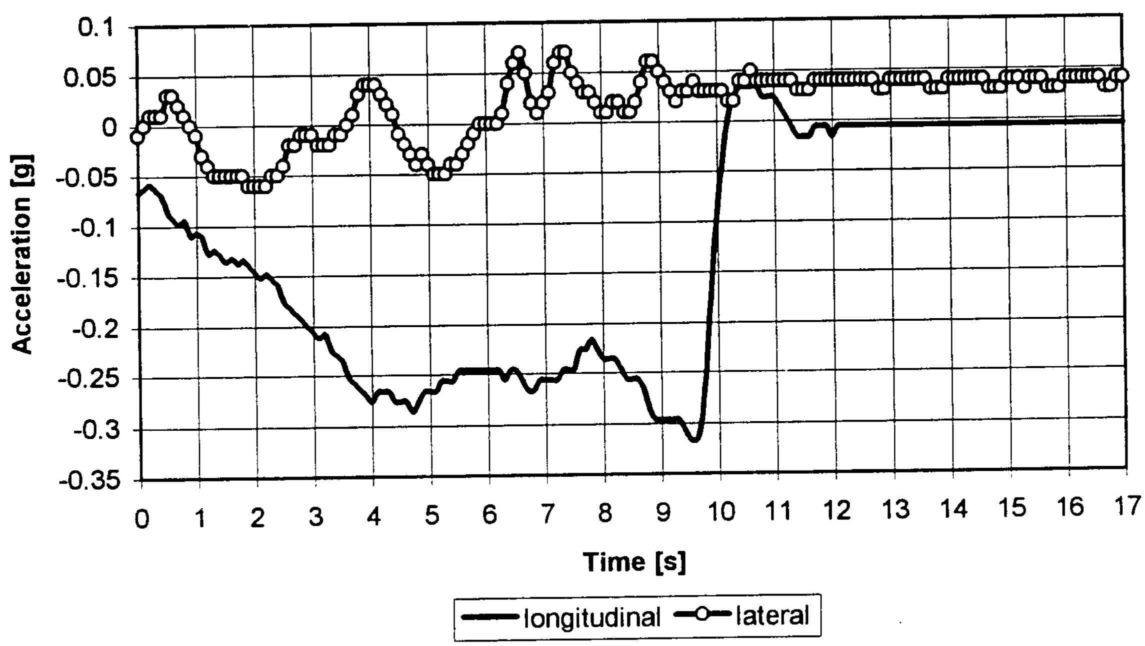

Figure 3.15 A Typical Deceleration Profile from the Ascension Lane Tests

Referring to Figure 3.15, at t = 0 seconds the test vehicle entered the ascension lane and immediately experienced a deceleration of approximately 0.06 g. As the vehicle ascended, the deceleration gradually increased to a fairly constant deceleration of 0.25 g. At t = 9.5 seconds, the vehicle experienced a sharp peak deceleration of 0.32 g. This peak value is the result of the driver applying the brakes firmly to prevent the vehicle from rolling backwards. As noted in the braking tests, a slight positive acceleration due to elastic and inertia effects is recorded at t = 10.5 seconds. The deceleration profiles obtained from different G-Analyst locations for each of the three ascension lane tests were fairly consistent and did not vary significantly in shape.

Descending Grade Arrestor Bed Test

A single run was made into the arrestor bed at an entry speed of 70 km/hr. The test vehicle entered the arrestor bed smoothly, with no sudden deceleration at the entrance. As the vehicle plowed through the arrestor bed, gravel was sprayed onto the entire width of the adjacent service lane. The test vehicle tracked a straight line through the arrestor bed, and no load shift was evident. A cloud of dust trailed the test vehicle but quickly dispersed. Tracks from the test vehicle were formed in the arrestor bed 8.9 m from the transition point between the end of the hard packed gravel entrance lane and the beginning of the arrestor bed. These tracks had an average depth of 200 mm (roughly half the depth of the arrestor bed). After the vehicle came to complete stop, it was inspected for damage. It was noted that the tractor undercarriage made slight contact with the gravel bed, but did not sink far enough to have front bumper contact. Gravel was sprayed into the undercarriage of the tractor and trailer, but the impact forces were not sufficient to cause visible structural damage. Slight discolourations and paint chips on the undercarriage surfaces of the tractor, however, were present as a result of the gravel spray. Table 3.4 summarizes the depth of penetration into the arrestor bed for each axle.

Table 3.4 Depth of Penetration for Each Axle in the Arrestor Bed at Final Rest

	Driver Side	Passenger Side
Front Axle	310 mm	280 mm
Trailing Axle 1	296 mm	330 mm
Trailing Axle 2	290 mm	280 mm
Trailing Axle 3	210 mm	280 mm
Trailing Axle 4	180 mm	340 mm

Table 3.5 summarizes the maximum decelerations experienced and the stopping distance required. Maximum decelerations were in the range of 0.43 g and were consistent throughout the instrumented locations on the test vehicle.

Table 3.5 Maximum Deceleration and Stopping Distance in the Arrestor Bed

Arrestor Bed Test				
Entry Speed [km/hr]	70			
Location of G-Analyst	Cab	Hitch	Right Rear	Left Rear
Maximum Deceleration [g]	0.43	0.43	0.42	0.43
Stopping Distance [m]	100.25			

Extraction of the vehicle was straightforward, and simply involved a tow truck (note that the tow truck used is one of about four existing in North America due to its large towing capability) winch pulling the test vehicle forward and out of the arrestor bed. No wooden planks were required to be laid under the wheels to ease the removal process. Also, it was unnecessary to remove the load from the trailer. It should be noted that the hold down anchors along the service lane were spaced too far apart, and none were in close proximity to where the test vehicle was stopped. As a result, the hold down anchors could not be utilized. Instead, metal claws, attached to the tow truck unit, were embedded into the asphalt of the service lane to provide anchorage.

Figure 3.16 illustrates a typical deceleration profile from the arrestor bed test:

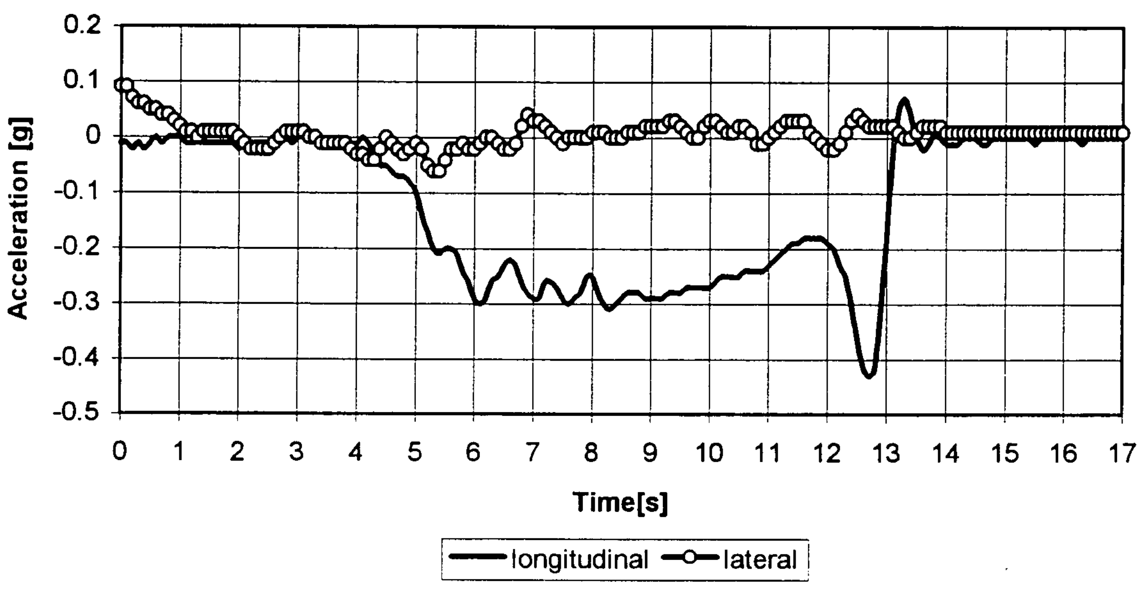

Figure 3.16 A Typical Deceleration Profile from the Arrestor Bed Test

At t = 4 seconds, the test vehicle entered the arrestor bed. A fairly constant deceleration of approximately 0.2-0.3 g was maintained thereafter. At t = 12.5 seconds, a sharp peak deceleration of 0.43 g is experienced and is the result of the driver applying the brakes. As noted in the braking tests, an slight positive acceleration is recorded at time = 13.5 seconds. The deceleration data obtained from each of the different G-Analyst locations for the arrestor bed test were consistent in both profile shape and deceleration magnitude.

Figure 3.17 Depth of Penetration in the Arrestor Bed

3.4 Discussion of Results

Braking Test

The braking test served as a good simulation of high deceleration rates experienced in the ascension lane and the arrestor bed test. The braking test achieved peak decelerations (i.e., 0.51 g) that were higher than those experienced in the ascension lane (i.e., 0.34 g) or the arrestor bed (i.e., 0.43 g). It indicated that the loads were secured properly, and put to rest any concerns that individual pieces of lumber on the top loads were going to shift forward. Although the load shifted slightly forward during this test, it was not significant to cause damage to the vehicle. The load shift, however, tensioned the webbing further (this became a factor that caused the load to shift backwards in the ascension lane test). Possible causes of the load shift can be attributed to improper dunnage, and a small gap that existed between the load and the front metal stakes. The dunnage consisted of smooth economy grade '2x4' lumber; rough dunnage would have been a better choice because it would have provided a higher coefficient of friction between the wooden deck and the load, thereby minimizing load shift

Ascension Lane Test

The ascension lane safely stopped the test vehicle without causing injury to the driver or damage to the test vehicle itself. Deceleration forces were well distributed, with no sharp deceleration experienced at the entrance. Average deceleration rates were in the range of 0.25 g. There is concern that runaway trucks on the ascension lane may roll backwards once they come to a complete stop. The driver of the test vehicle noted that very little time existed between the time when the vehicle came to a complete stop and the time it began to roll backwards. A bed of arresting material or similar facility installed near the top portion of the ascension lane may be a possible solution to prevent trucks from rolling down. Another possible solution may be to reduce the grade of the top portion of the ascension lane slightly.

Stopping distance required on the third test was higher because no brakes were applied during the ascent, as in the case of the first two tests.

Descending Grade Arrestor Bed Test

The arrestor bed was more efficient in stopping the test vehicle when compared to the ascension lane. No sudden deceleration was experienced at the entrance due to the gradual, rather than abrupt, increase of depth of the pea gravel over the first ten metres of the bed. The potential for structural damage to a truck entering the arrestor bed is minimal. Both the stopping time and distance required in the arrestor bed were shorter than those recorded in the ascension lane. The trade off, however, is that slightly higher average deceleration rates were experienced in the arrestor bed. Average deceleration rates were in the range of approximately 0.30 g, decreasing to approximately 0.2 g prior to final rest. Peak deceleration forces were achieved quickly shortly after entering the arrestor bed. The effectiveness of the hard packed gravel entrance lane prior to the arrestor bed, according to the driver, was questionable. The driver indicated that it was difficult to keep the vehicle in a straight line in this section

Figure 3.18 Final Resting Position of Truck in Arrestor Bed

Both the ascension lane and the arrestor bed have been shown to be an effective means of decelerating runaway trucks safely and efficiently. Deceleration forces experienced in these runaway lanes with an entry speed of 70 km/hr, were actually lower than those experienced by the test vehicle applying the brakes firmly on asphalt at 40 km/hr (< 0.5 g).

3.5 Summary

The ascension lane provided a more gradual increase in deceleration forces, compared to the arrestor bed. Average deceleration was also lower in the ascension lane, but as a result, the distance required to stop the vehicle was increased. Both types of runaway lanes did not induce any visible structural damage to the test vehicle, nor did it cause any significant load shift. Difficulty may arise with the ascension lane in that no arrestor bed or a similar facility exists to prevent runaway trucks from rolling backwards once they come to a complete stop. Extraction of the vehicle in the arrestor bed was easier and less time consuming than previously thought. All that was required was a very large tow truck winch pulling the test vehicle forward and out of the bed.

Figure 3.19 Truck Effectively Stopped by an Arrestor Bed with no Load Shift

4.0 Design Considerations for Runaway Lanes

4.1 Types of Runaway Lane

There are several different types of runaway lane facilities, and are primarily distinguished by the type of vehicle deceleration mechanisms they utilize. Two common methods of vehicle deceleration mechanisms include 1) an arresting material, such as gravel or sand, to provide rolling resistance to the tires, and 2) a steep upgrade to dissipate the kinetic energy of a runaway vehicle into potential energy through gravity forces. The most commonly known types of runaway lane include ascension lanes, sand piles, and arrestor beds. A relatively new type of runaway lane facility utilizing dragnets has also emerged, although its use for this type of application has been very limited. Regardless of the type, an optimal runaway lane facility should have the capability to stop a runaway vehicle, with minimal damage to the vehicle and occupants, while maintaining low installation and maintenance costs.

Ascension Lanes

Ascension lanes or gravity lanes are the most common type of runaway lane in existence because they are relatively easy to construct. For example, in British Columbia, early forms of ascension lanes utilized old logging roads or roads that were abandoned in realignment projects. Their location, however, is usually dictated by the surrounding topography; they are limited to areas that have a significant upgrade in close proximity to a downgrade. An ascension lane consists of either a hard surface or layers of loose gravel, or uncompacted sand and ascends a fairly steep grade. It slows a vehicle by relying primarily on the force of gravity acting opposite to the vehicle direction of travel. As the vehicle ascends, its kinetic energy is transformed into potential energy. Thus, in order to provide sufficient retarding force, the ascending grade must either be severe, with 25 to 30 % grades not being uncommon, or very lengthy. In British Columbia, there are at least seven ascension lanes with grades in excess of 25 %. As a consequence of the steep upgrades involved, some ascension lanes contain a small aggregate arrestor bed near the top to prevent runaway vehicles from rolling backwards and jackknifing. As a final preventative measure, attenuation barrels may be installed at the end of the lane. Figure 4.1 illustrates the various components of an ascension lane.

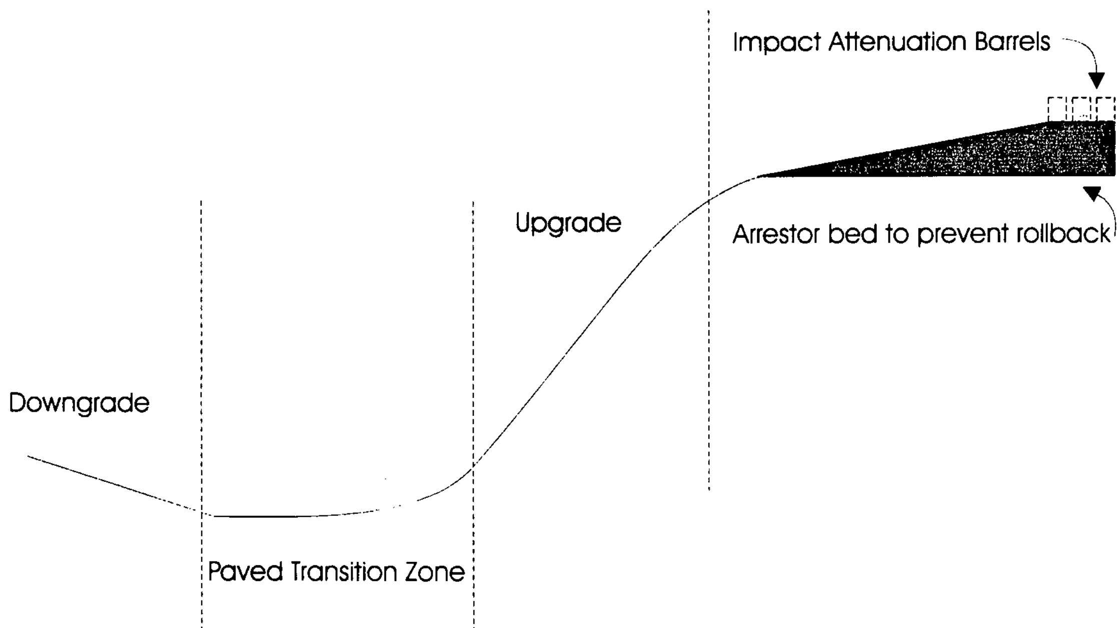

Figure 4.1 Components of an Ascension Lane

Sand piles

Sand piles or mounds are the easiest and least expensive type of runaway facility to construct, but performance is less than adequate due to problems associated with maintaining the bed to a proper standard. Sand piles consist of piles of loose sand placed along the roadside with the top surface of the material being either level with the roadway, slightly ascending, or arranged in transverse berms. They operate by dramatically increasing the rolling resistance to the vehicle's tires through a plowing, bulldozing effect. Construction of these facilities are discouraged because of the abrupt deceleration rates they can impart on runaway vehicles. Because the retarding forces are concentrated on the front axles and are not distributed among the trailing axles, a force imbalance occurs. This results in possible load shifting or jackknifing of the runaway vehicle. There are also maintenance concerns: sand compacts over time, and if not loosened very frequently, loses its effectiveness.

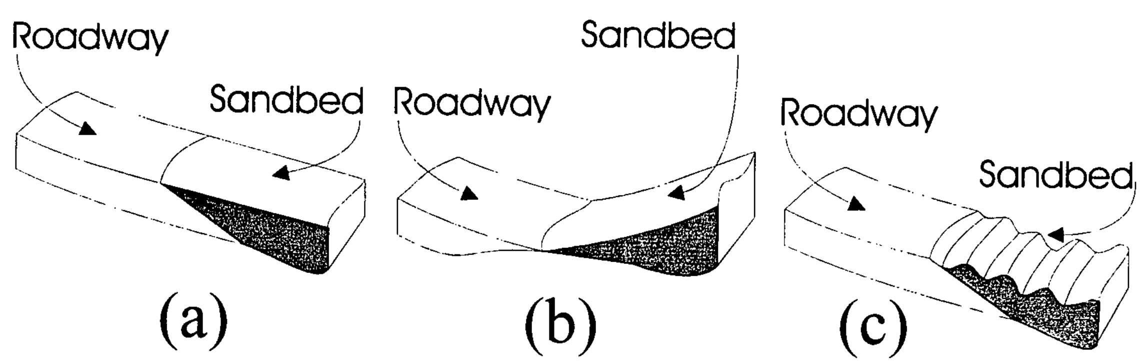

Figure 4.2 Sand Piles: (a) level with the roadway, (b) ascending, (c) transverse berms

Arrestor Beds

Arrestor beds are the most effective type of runaway lane facility because it is possible to locate them where topography prohibits the construction of other types of runaway lane facilities. Arrestor beds can be constructed on either a level, up or downgrade. In addition, they have been proven to be effective at very high speeds. These facilities are typically shorter than ascension lanes and do not require an upward slope to decelerate a vehicle. The principle retarding mechanism lies in the aggregate bed which provides a combination of rolling resistance and a plowing effect. As the tires penetrate the surface of the gravel, the vehicle's kinetic energy is dissipated through 1) the transfer of momentum through the gravel spraying, 2) aggregate compaction, 3) bulldozing effects, and 4) the side shearing, abrasion and fracturing of the aggregate. Another advantage of arrestor beds is that their performance is relatively independent of vehicle mass.

Figure 4.3 Runaway Truck Embedded in an Arrestor Bed

Dragnets

A new type of vehicle retarding system commonly referred to as dragnets has recently been evaluated for its application for runaway lane facilities [33]. This system was originally developed for the United States Navy to stop aircraft on aircraft carriers. A dragnet system consists of anchor posts, energy-absorbing reels of steel tape, and a net assembly (see Figure 4.4). Dragnets are not considered to be an effective stand alone system for several reasons. First of all, dragnets are less cost effective compared to an arrestor bed type facility designed to serve the same purpose. Secondly, the length required to stop a runaway vehicle using a dragnet system would be greater than an arrestor bed, if deceleration forces are to be kept at relatively comfortable levels. In addition, the anchor posts that are needed to hold the nets in place pose a safety hazard because runaway vehicles may strike them. More importantly, dragnets are mass and speed dependent; they are designed to dissipate a certain amount of kinetic energy for a given entry velocity and vehicle mass. Therefore, a dragnet system designed for a heavy vehicle traveling at a very high speed may be hazardous to a vehicle of a lesser mass traveling at the same speed. Finally, there have not been any full scale tests to evaluate the performance of dragnets for heavy trucks (greater than 63,500 kg) entering at very high speeds (greater than 100 kph). Dragnets, however, may be effective as a supplemental system, for example, at the end of an arrestor bed, where vehicle speeds will not likely be as high and the consequences of striking an anchor post will not be as severe. Figure 4.4 illustrates the various components of a dragnet system.

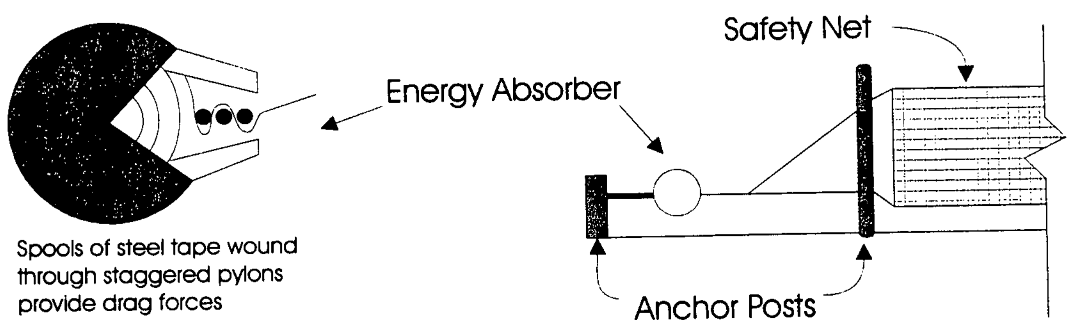

Figure 4.4 Components of a Dragnet System

4.2 Delineation and Approach to Runaway Lane Entrances

The type of delineation available at or near the entrance of the runaway lane facility is very important in terms of governing the proper use of the facility. Proper delineation should be installed such that the following criteria be satisfied. First of all, delineation should clearly indicate the entrance of the runaway lane facility to minimize the possibility of it being missed by a runaway truck. Secondly, delineation should make the facility 'inviting' or simpler for operators of runaway trucks to enter by providing positive guidance and a perceived sense of safety. At the same time, however, delineation should discourage the unauthorized use of the runaway lane facility, such as motorists mistaking the runaway lane as a rest area or exit ramp.

The access of the runaway lane must be clearly indicated with proper signage and must be located such that sufficient sight distance is provided well in advance to allow operators of runaway vehicles time to react. These advisory signs must be placed reasonably frequently before the entrance of the runaway lane facility. Figure 4.5 provides an example for sign layout for a runaway lane. Overhead signs indicating the entrance of the runaway lane and adjacent service lane, if present, are ideal.

To guide the operator of a runaway truck into a runaway lane, road-edge delineation markers on both edges of the lane may be helpful in this regard. Flexible reflective delineator posts are recommended especially when the lane is covered with snow or when the lane is used at night.

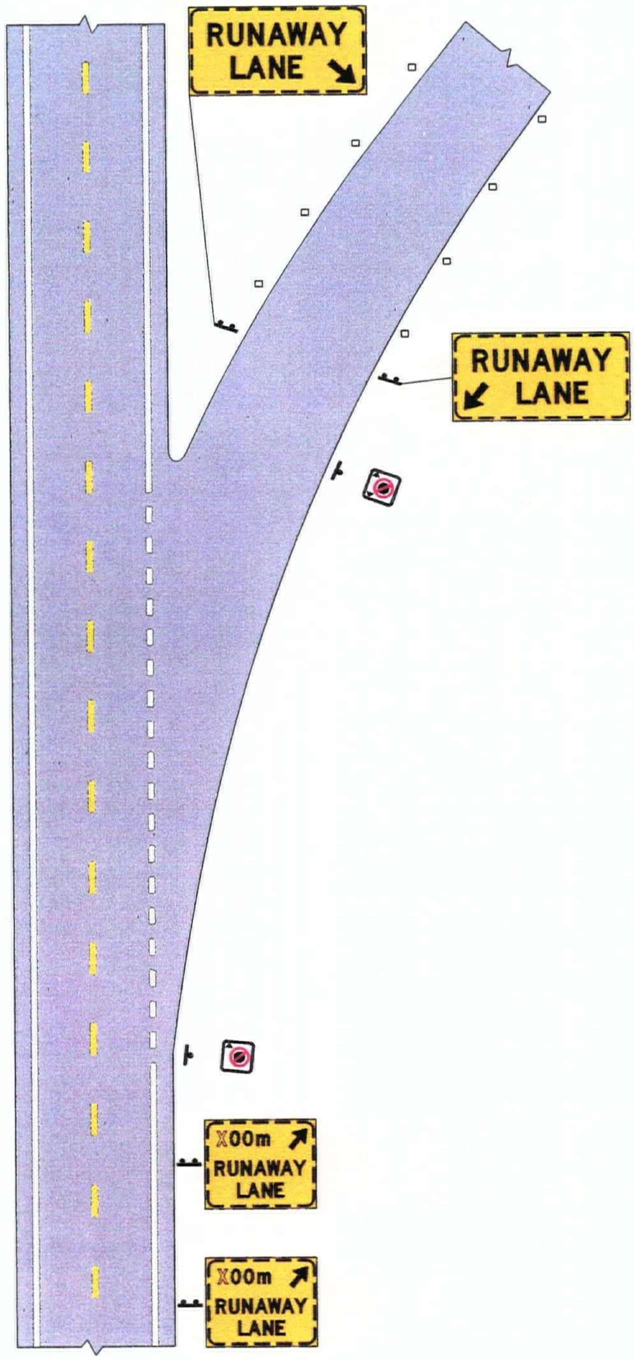

Figure 4.5 Example of a Sign Layout for a Runaway Lane

These delineator posts may also be used as a quick reference to gauge the approximate distance the runaway truck has traveled to measure the effectiveness of the facility. In addition, delineator posts may be used as a reference for correct depth of gravel in arrestor beds when it is being leveled during maintenance. For ascension lanes with severe grades (in excess of 20 %), it is suggested that the lane, in addition to reflective delineation posts, be supplemented with concrete barriers on the downslope edge. These barriers would provide protection for runaway trucks from rolling off steep downslope embankments, and at the same time provide a sense of perceived safety to truck drivers who are contemplating on using the lane.

Illumination of the approach and runaway lane is beneficial and placement of overhead lights at the entrance should be considered if costs justify.

It is equally important to properly sign the facility to prevent unauthorized use. This is especially important in the case of arrestor bed type runaway lanes where vehicles can become entrapped for an extended period of time and possibly be struck by a runaway truck legitimately using the facility. There is also the extra expense to smooth wheel tracks caused by drivers mistaking the arrestor bed as a rest area or exit ramp. Therefore, it is imperative that any type of delineation should not inadvertently attract a motorist into the runaway lane area. It has been suggested in the literature that red delineator reflective markers be used instead of the more common white or yellow markers. A more novel approach, as implemented by the Quebec Ministry of Transportation involves painting the entire entrance of the runaway lane with a red and white checkerboard pattern (see Figure 4.6). Pavement markings are an effective means to minimize parking or confusion about the main travel lane direction. In addition, NO PARKING or NO STOPPING signs should be located at the entrance to discourage the casual use of the paved entrance.

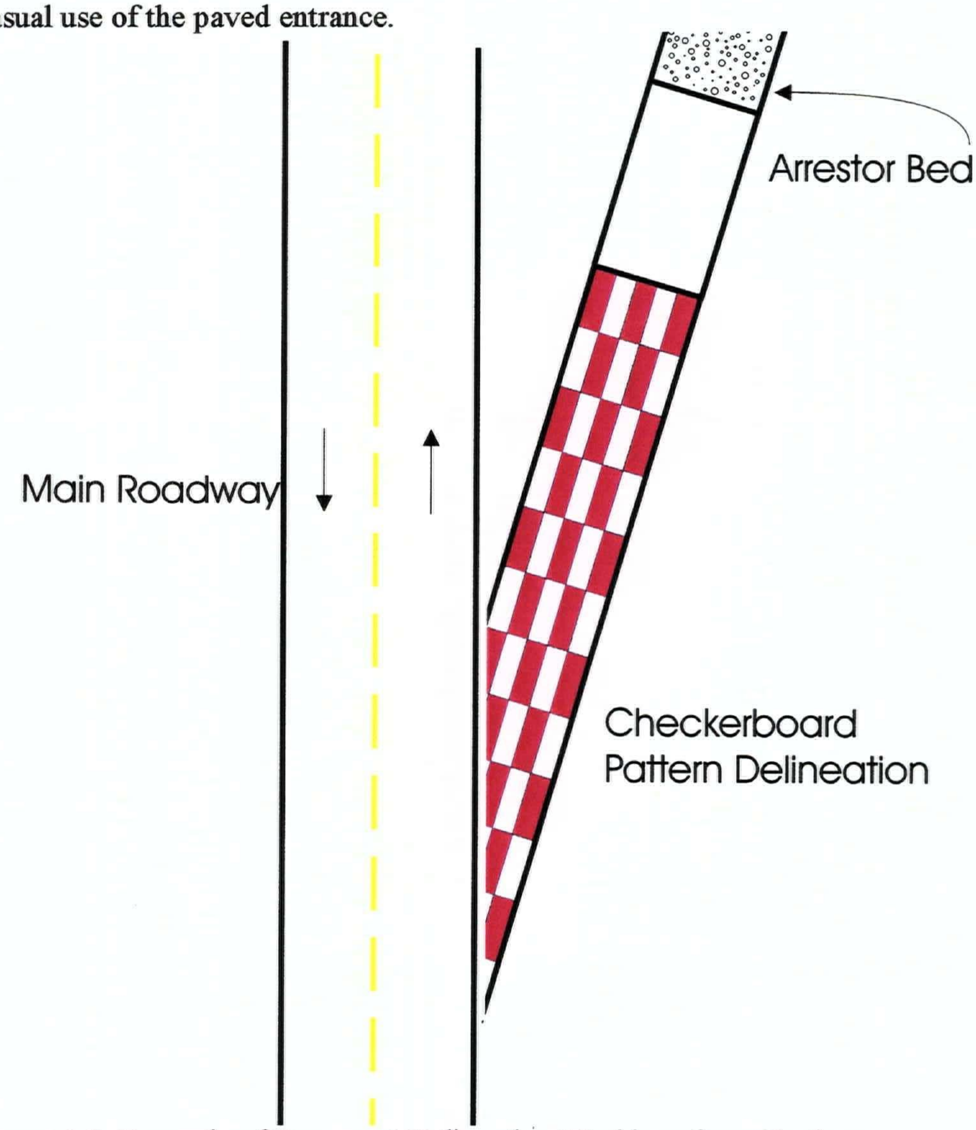

Figure 4.6 Example of Pavement Delineation Markings from Quebec

The approach of a runaway facility should be designed such that the access is apparent and in the full view of the operator of the runaway vehicle. It should leave the mainline at the flattest angle possible to permit vehicles traveling at a high rate of speed to enter the runaway lane safely. A runaway vehicle should not be required to encroach the outer or opposing lane in order gain access to the entrance. Typical departure angles are in the range of 3 to 5 degrees. Alternatively, the following equation may be used to determine whether the exit radius of curvature is adequate to accommodate the speed of a runaway vehicle.

$$R = \frac{v^2}{12.96g(e + f_s)}$$

where:
R = radius of curve [m]
v = speed of the vehicle at entrance to runaway lane [km/hr]
g = acceleration due to gravity 9.81 [m/s^2]
e = rate of superelevation
f_s = coefficient of side friction

An exit lane that parallels the mainline, however, is ideal, to allow out-of-control vehicles more space to maneuver. A hard paved entry should also be provided immediately before the runaway lane. This paved entrance serves two purposes: 1) it provides the driver preparation time to steer the vehicle correctly before actual deceleration begins, and 2) it facilitates the front tires of the vehicle to enter the arresting material squarely and simultaneously to minimize yaw motion.

4.3 Design Entry Speeds

The design entry speed can be defined as the most probable speed that a runaway vehicle will attain at a certain location along the downgrade where a runaway lane facility is to be located. There are essentially two schools of thought in terms of determining a design entry speed for the calculation of a minimum required length for a runaway lane facility. The first methodology involves a combination of experience and engineering judgment. In this method, a single speed is recommended, and encompasses all road segments, regardless of site specific conditions. These recommended speeds have arisen primarily from site observations and experience. In many cases, these recommended design speeds are overly conservative and results in over designing of runaway lane facilities. For example, the AASHTO *Green Book* recommends a design entry speed of between 80 to 90 mph (129 to 145 kph), which is consistent with several other state highway authorities in the United States. The state of Colorado suggests using an even higher design speed of 100 mph (161 kph). However, the AASHTO *Green Book* further states that "speeds in excess of 80 to 90 mph will rarely, if ever, be attained," confirming that the recommended speeds may indeed be conservative.

The second methodology, or the theoretical approach, uses analytical methods to derive a design speed. The most simplistic of these analytical methods is to equate the total kinetic and potential energy of the vehicle without taking into account the retarding effects of air resistance, chassis friction, and road friction. This energy summation would be in the following form:

$$\frac{1}{2}mv_0^2 + mgh = \frac{1}{2}mv_f^2 \quad \text{or} \quad v_f = \sqrt{v_o^2 + 2gh}$$

where:

m = weight of vehicle [kg]
v_0 = initial speed [m/s]
g = gravitational constant 9.81 [m/s^2]
h = difference in elevation [m]
v_f = final speed [m/s]

The final speed predicted by the above equation, in many instances, however, is not practical because it is usually higher than the terminal speed of a runaway truck. Also the equation cannot take into account elevational fluctuations between the top and bottom of the downgrade.

A more refined analytical approach taking into account resistance factors of the truck and surrounding topography was developed by Stanley [43] and is represented by the following equation:

$$v = 5.469\sqrt{0.03343v_0^2 - H - KL - 0.000016v_mL - \frac{0.0012v_n^2}{W}}$$

where:

v_o = initial speed [miles/hour]
H = difference in elevation [feet]
K = constant incorporating surface friction and speed-independent portion of mechanical loss: 0.01675 for pavement and 0.26175 for gravel bed
L = length of grade [feet]
v_m = average of v_o and v
F = frontal area of truck [feet^2]
v_n^2 = average of v_o^2 and v^2
W = truck weight [lb]

Stanley developed the above equation to determine the speed of a truck at any point on a grade based on energy summation principles. A spreadsheet program was created based on this equation (see Figure 4.7 for a sample output). Note that when using the Stanley program, lengths should be limited to 150 to 300 m segments for better accuracy. In general, the speed prediction equation is accurate up to a 8 km segment for a 3 % downgrade or a 1 km segment for a 10% downgrade, after which resistance factors do not limit speed greatly and terminal velocity approaches quickly.

As a final note, it would not be feasible to use a design speed that has only a remote chance of being attained by a runaway truck. Such situations exists when the geometrics of the road segment limit vehicle speed. It would not be practical to use a speed of 60 kph, for example, when a horizontal curve limits speed to 40 kph. Assuming that the upper limit of lateral acceleration to prevent a truck rollover is between 0.30 to 0.35 g, the following equation can be used to determined a maximum cornering speed:

$$v = \sqrt{agR}$$

where :

a = maximum lateral acceleration [0.30 to 0.35 g]
g = gravitational constant 9.81[m/s^2]
R = radius of curvature [m]

In the design of runaway lanes, one must select a design entry speed that will not likely be exceeded, but at the same time, the designer must not be overly conservative as the length of the runaway lane required and hence the costs, increases with increasing entry speeds. An analytical approach incorporating topographical and vehicle parameters seems to a reasonable approach in terms of predicting a suitable design entry speed because it can take into account site specific conditions. The Stanley equation shows promise in this regard.

Stanley Equation for Determining Speed of Truck on a Downgrade

User Specified Input

Frontal Area	9.29	[m^2]
GVW	63500	[kg]
Initial Speed	0	[km/hr]
Terminal Velocity	130	[km/hr]

Road Segment	Grade [%]	Length [m]	Height [m]	Stanley Speed [mi/hr]	Cumulative Distance [km]	Stanley Speed [km/hr]	Elevation [m]
0			0	0.00	0	0.00	160.00
1	-8	200	-16	34.93	0.2	56.04	144.00
2	-8	200	-16	49.09	0.4	78.76	128.00
3	-8	200	-16	59.78	0.6	95.91	112.00
4	-8	200	-16	68.66	0.8	110.16	96.00
5	-8	200	-16	76.37	1	122.52	80.00
6	-8	200	-16	81.03	1.2	130.00	64.00
7	-8	200	-16	81.03	1.4	130.00	48.00
8	-8	200	-16	81.03	1.6	130.00	32.00
9	-8	200	-16	81.03	1.8	130.00	16.00
10	-8	200	-16	81.03	2	130.00	0.00

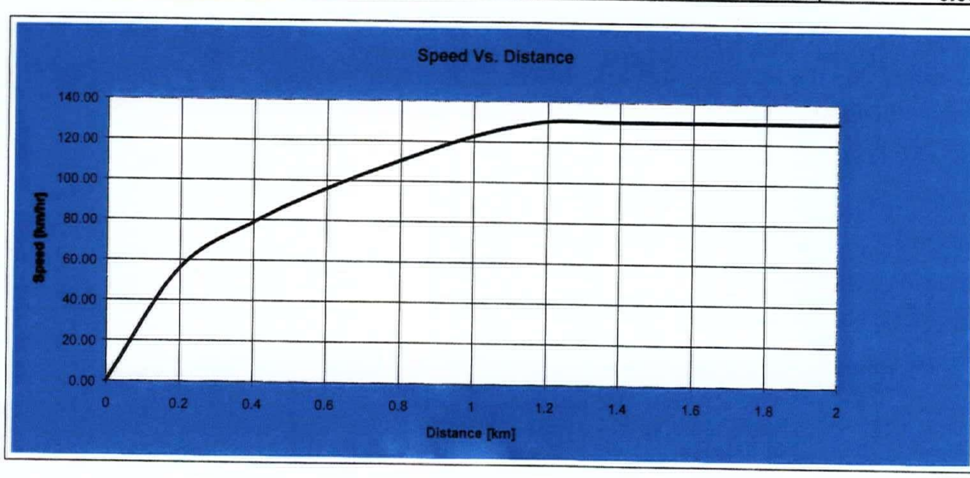

Figure 4.7 Sample Output from Spreadsheet Program to Predict Downgrade Speeds

4.4 Width of Runaway Lanes

The width of a runaway lane facility is generally not dependent on the type (i.e., ascension or arrestor bed). It is however, directly related to the ease of removal of a runaway vehicle. In other words, it is related to the type of backup measures available, and whether a runaway vehicle can be removed expediently before another runaway occurs and requires the use of the facility. It is well documented in the literature that, ideally, the width of runaway lanes should be able to accommodate at least two vehicles to use the facility at the same time. However, this may be infeasible, due to topography limitations, and ultimately cost. It is better to provide for quick turn around times by developing faster removal procedures, such as implementing better arrangements with local tow truck companies.

In British Columbia, ascension lanes have average widths ranging from as low as 3 metres, to as high as 12 metres. Arrestor beds have average widths ranging from 4 to 6 metres.

It is suggested that any current and new ascension lanes be upgraded to a width of at least 5 metres, which leaves approximately 1 metre of free space on each side of a typical truck. Widths accommodating more than two vehicles concurrently are generally not used in ascension lanes because vehicles do not become entrapped, and thus do not occupy the facility for an extended period of time. In the case where the ascension lane meanders considerably, the width should be increased accordingly to allow the runaway truck more maneuverability, or alternatively, alignment should be straightened as much as possible. A width of 6 metres for an arrestor bed type runaway lane is generally considered adequate but must be accompanied by an adjacent service lane of similar width to facilitate prompt removal of an entrapped vehicle. The service lane also acts as a buffer zone to prevent gravel sprayed from the arrestor bed from reaching the highway, and creating a potential hazard to passing motorists. It was noted during the full scale testing of arrestor beds in Merritt, B.C (see previous chapter) that the gravel sprayed the entire width of the service lane which had an average width of 3.6 metres.

The above suggested widths are capable of handling only one runaway vehicle at any particular time.

4.5 Surface Material for Arrestor Beds

Specific Gradations and Associated Deceleration Rates

The most preferred type of surface material for use in arrestor beds is uniformly graded pea gravel. The benefits of pea gravel include good drainage properties, resistance to freezing and compaction, and the ability to provide adequate deceleration rates. The use of sand as an arresting material is not recommended because it is not practical to maintain a sand bed in a dry and uncompacted condition at all times. In addition, sand beds are prone to freezing, and are not suitable for the cold climate conditions of B.C..

In general, the gravel used in arrestor beds should adhere to the following characteristics for optimum performance. The gravel should be predominately single sized and uniform, approximately 12-25 mm in diameter. This criteria serves to minimize moisture retention and problems associated with freezing. The uniformity of the gravel can be measured by the uniformity coefficient D_{60}/D_{10}, where D_{60} represents the gravel diameter of which 60% of the gravel weight is finer and D_{10} is the corresponding value at 10% finer. A uniformity coefficient smaller than two is considered to be uniform. The gravel should be clean and free of fines to minimize maintenance due to compaction. Smooth, rounded gravel, as opposed to crushed angular gravel, is preferred to maximize void spaces. Void spaces are essential for proper drainage. In addition round gravel minimizes interlocking, and allows tires of the runaway vehicle to sink in more easily, facilitating higher retarding forces. The deceleration provided by gravel is dependent on the specific gradation, and to a certain extent, the depth of material. Typical deceleration rates provided by gravel range from approximately 0.2 g to 0.4 g. The following is a sample of specific gradations and associated deceleration rates currently in use in arrestor beds and may be used as a reference when selecting the material and depth for design purposes.

The Siskiyou Summit Negative Grade Arrestor Bed in Oregon uses the gradation in Table 4.1 and has been shown to be quite successful in providing adequate and safe deceleration levels. In addition, no problems with freezing and solidification have been encountered since construction. The expected deceleration rates in Table 4.2 are extrapolated to entry speeds of 80 mph (129 kph) based on test results from actual tested entry speeds up to 60 mph (97 kph). The deceleration rates are based on a gravel depth of 450 mm.

Table 4.1 Gradation of Arrestor Bed Gravel at Siskiyou Summit

Size	Percent Passing
19 mm (3/4 in)	100
12.7 mm (1/2 in)	0-15
6.4 mm (1/4 in)	0-5

Table 4.2 Expected Deceleration Rates from Siskiyou Summit Arrestor Bed

Vehicle Type	Deceleration [g]
empty 2-axle	0.25
empty 5-axle	0.24
loaded 2-axle	0.32
loaded 5-axle	0.26
Mean Deceleration (all vehicles)	*0.27*
Standard Deviation	*0.036*

The Mt. Vernon Canyon gravel arrestor bed in Colorado has also demonstrated successful performance and the specific gradation is outlined in Table 4.3.

Table 4.3 Gradation of Arrestor Bed Gravel at Mt. Vernon Canyon

Size	Percent Passing
25.4 mm	100
9.53 mm	90-100
#4 (4.76 mm)	0-15
#8 (2.38 mm)	0-5
#200 (0.074 mm)	0-1

Expected deceleration from this type of gravel is estimated to be 0.32 g with a standard deviation of 0.14 g. The deceleration values were back-calculated using the FHWA equation and data from 45 recorded entries into the gravel arrestor bed during a three year period and is based on a depth of the gravel ranging from 450 to 600 mm. It should be noted that the state of Colorado recommends a deceleration value of 0.2 g to be used for design purposes to incorporate a factor of safety.

After extensive testing of existing arrestor beds in Pennsylvania [48], the Pennsylvania Transportation Institute (PTI) recommends a rounded river gravel with a mean diameter of 12 mm. The expected deceleration of this material is 0.36 g with a standard deviation of 0.05 g, and is based on full scale tests using a tractor-trailer configuration and a gravel depth of 914 mm.

To address the concerns of gravel arrestor beds freezing solid in cold climates Derakhshandeh [17] studied the behavior of various aggregate distributions in arrestor beds prone to freezing in the State of Colorado. The study recommended the following gradation for cold weather conditions. The gradation of Table 4.4 should allow for proper drainage and prevent the bed from freezing solid.

Table 4.4 Gradation of Arrestor Bed Gravel Resistant to Freezing Conditions

Size	Percent Passing
50.8 mm (2 in)	100
25.4 mm (1 in)	25
19.1 mm (3/4 in)	10
12.7 mm (1/2 in)	5
9.52 mm (3/8 in)	0-5

In addition to the general shape and size requirements, appropriate tests should be conducted for the abrasion resistance and durability of the aggregate (e.g., L.A. abrasion test) to avoid costly replacement costs. A geotechnical engineer familiar with these tests should be consulted for this purpose.

The Effect of British Columbia's Higher Legal GVW Limit and Increased Number of Axles on Published Deceleration Rates

The above noted gravel gradations and associated deceleration rates apply only for 5-axle trucks weighing up to 36,000 kg (80,000 lb). Since almost all the documented sources for full scale testing with the above gravel gradations has been performed to date on vehicles weighing less than 36,000 kg and having less than or equal to 5-axles, one may question whether the associated deceleration rates may be applied for trucks in British Columbia which can weigh almost twice as much and have up to 9-axles.

It has been well documented that vehicle load is not significant on the stopping distance in gravel arrestor beds (an increase in momentum is counteracted by an increase in drag forces caused by deeper penetration into the gravel). Therefore, it may be concluded that the first issue of the larger permissible GVW of British Columbia trucks have a minor effect on deceleration rate and thus the stopping distance. However, the increase in the number of axles *may* have an effect on deceleration rate according to Booze [11] who noted that

> Subsequent passes of following wheels through a rut made by the leading wheel significantly decreases the retarding effectiveness of the medium...This must be due in part to the packing of the material by the leading wheel reducing the penetration of subsequent wheels and thereby the amount of material the following wheels will displace. Third and fourth passes did not result in significant further reduction of braking action.

From model tests conducted by Booze, it was shown that trailing axles produce approximately 25% less resistance than the front axle. Since all the deceleration values in the literature are tested for vehicles up to 5 axles, how do we account for vehicles such as an 8-axle B-train? One method proposed is to take the existing deceleration data for 5 axles and extrapolate an average deceleration value for 8 axles.

For example, according to the tests at the Siskiyou Summit arrestor bed, a five axle truck has an average deceleration rate of 0.26 g. Assuming that the trailing axles have a 25% reduction in deceleration forces, the distribution of deceleration forces among the axles are as follows:

Table 4.5 Distribution of Deceleration Rates Among Axles

Axle	Deceleration [g]	Deceleration [g]
1	a_f	0.33
2	0.75 a_f	0.24
3	0.75 a_f	0.24
4	0.75 a_f	0.24
5	0.75 a_f	0.24
all	Average	0.26

where a_f= deceleration rate on front axle

Using the above distribution of deceleration rates, an 8-axle vehicle would have an average deceleration rate of 0.25 g. It can be concluded, therefore, that an additional 3 axles (an increase from 5 to 8 axles) does not significantly reduce the average deceleration rate greatly.

As an aside, there has been only one documented source to date for full scale testing of trucks weighing more than 60,000 kg and having more than 5 axles. In mid 1994, Main Roads, in Western Australia conducted full scale trials to assess the performance of different arresting materials and bed configurations to control and stop vehicles weighing up to 79,000 kg. In their preliminary report [32], it was confirmed that it was possible to safely stop a 9 axle B-train traveling at high speeds in an arrestor bed without damage. In a particular trial, the test vehicle used weighed 62,200 kg and entered the arrestor bed with speeds varying from 60-112 kph. The range of stopping distances were between 42.8-110.2 m, with a range of mean deceleration between 0.33-0.45 g. The arrestor bed was filled with an artificial lightweight aggregate known as "LYTAG", to a depth of 400 mm. The aggregate is a nominal 6 mm size and is commonly used in arrestor beds in the United Kingdom. Similar tests were conducted using a local river gravel of the same size, and reproduced similar deceleration rates.

From these tests, one may conclude that deceleration forces for rounded aggregate are fairly consistent, and do not change significantly, even for truck configurations with more than five axles. It can be assumed, therefore, that deceleration rates derived from testing of 5 axle trucks, may be applicable for the larger configuration trucks currently existing on British Columbia highways.

Recommended Taper for Arrestor Beds

To minimize abrupt deceleration forces on the front axle of runaway vehicles entering an arrestor bed, it is essential that the depth of aggregate be gradually increased to the maximum depth over a reasonable distance. If these abrupt deceleration forces are not minimized at entry, the possibility of axle bolt or king pin shearing at the fifth wheel may result, causing loss of control and perhaps jackknifing. Load shifting, causing harm to the driver and occupants may also result.

Suggested tapers vary among the literature. For example, the Colorado Design Guide [26] recommends a gradual increase to a minimum depth of 457 mm in 30 m. AASHTO [6] suggests that the depth of the bed should be "tapered from 3 inches at the entry point to the full depth of aggregate in a minimum of 100 feet." Allison et al. [5] describes an arrestor bed used in New York having a taper rate of 25:1 without causing abrupt deceleration problems. It was concluded that very little, if any deceleration occurred upon entry for about 1 second. Arrestor beds in Arizona [18], also with taper rates of around 25:1, had similar findings. Although suggested taper rates vary, there seems to be a general consensus that the gradual increase in depth of the aggregate should occur in the first 30 metre or so. Interestingly, an arrestor bed in Pennsylvania, with an initial depth of 457 mm (the full depth of some arrestor beds) did not cause abrupt decelerations problems during testing [48]. With this in mind, the initial depth of the aggregate bed may not necessarily start at zero, to take advantage higher deceleration forces of a deeper bed much earlier, without causing abrupt deceleration problems. A taper in the aggregate bed may also be useful for extraction purposes, and may be the reason why some arrestor beds have tapers at the beginning and end of the bed.

Aggregate Depth

The resistance provided by the gravel is somewhat dependent on the depth. Tests conducted by Booze et al. [11] found that the resistance of the gravel seems to level off at a depth of 457 mm. This leveling off trend, however, is only substantiated by one data point. Tests conducted by the Pennsylvania Transportation Institute found that any depth less than 762 mm would cause a vehicle in the arrestor bed to porpoise significantly causing uncomfortable levels of vertical acceleration. Therefore, it was suggested that the depth of gravel be at least 914 mm. Brown [13] conducted tests in Australia and found that depths greater than 450 mm caused damage to the undercarriage of the test vehicle. Later tests using depths of 350 mm posed no undercarriage damage problems to the vehicle.

AASHTO and the state of Colorado recommend a minimum depth of 305 mm and 457 mm respectively.

Tests conducted by the author (see previous chapter) using an arrestor bed with a depth of 450 mm effectively stopped a fully loaded (36,000 kg) 5 axle tractor-trailer without causing undercarriage damage or porpoising as described by tests conducted by PTI. Average depth of penetration of the tires at rest was approximately 300 mm substantiating AASHTO's recommended minimum depth.

To strike a balance between a depth for maximum effectiveness, while using the minimum amount of gravel, a depth of approximately 450 mm seems optimal. If a larger depth is used, undercarriage damage may occur, and as noted by Booze, deceleration rates are not greatly increased beyond this depth. On the other hand, if a smaller depth is used, then maximum deceleration forces are not achieved and longer arrestor bed lengths may be required.

4.6 Design Equations to Determine Required Length for Runaway Lanes

Design equations, both theoretical and empirical, have been developed as early as the mid 1940's to estimate the length required by a runaway lane to bring a runaway vehicle to a stop. Taragin [44] was the first to introduce such an equation. This equation, however, was initially developed to analyze the effect of grade on the speed of motor vehicles. Taragin's equation considered the change in kinetic energy, energy developed by the engine, energy to overcome the grade, and tractive resistance. The equation has the form:

$$\frac{1}{2}\left(\frac{W}{32.2}+k_n\right)\left(v_1^2-v_2^2\right)(1.47)^2+(TE)(L)=(W)(g)(L)+(W)(f)(L)$$

Solving for L:

$$L=\frac{1.075\left(\dfrac{W}{32.2}+k_n\right)\left(v_1^2-v_2^2\right)}{W(g+f)-TE}$$

Where:

W = gross weight of the vehicle [lb].
k_n = mass equivalent constant for the gear in which the vehicle is operating, which is a factor to compensate for the change in the kinetic energy of the rotating parts when accelerating or decelerating.
v_1 = initial speed of the vehicle [mph]
v_2 = final speed of the vehicle [mph]
TE = tractive effort for average speed during interval [lb]
f = coefficient of tractive resistance [lb/lb GVW]
g = grade

The Federal Highway Administration (FHWA) later adopted and modified this equation to predict stopping distances required in arrestor beds and ascension lanes. The tractive effort, mass equivalent constant for the gear in which the vehicle is operating, and the final speed of the vehicle were eliminated because it is assumed that a runaway vehicle would be out of gear and free rolling, and brought to a stop by the runaway lane. As a result, the weight term cancels out in the process. The modified equation in metric is as follows:

$$L = \frac{v^2}{254(R \pm G)}$$

Where:

L = distance required to bring vehicle to a rest [m]
v = entry speed of the vehicle [kph]
R = rolling resistance expressed as an equivalent % gradient divided by 100
G = grade divided by 100

The rolling resistance depends on the surface material and is summarized in Table 4.6.

Table 4.6 Rolling Resistance as a Function of Surface Material

Surface Material	Rolling Resistance [lb/1000 lb GVW]	Equivalent Grade [%]
Portland Cement Concrete	10	1.0
Asphalt Concrete	12	1.2
Gravel (Compacted)	15	1.5
Earth (Sandy, Loose)	37	3.7
Gravel (Loose)	100	10.0
Sand	150	15.0
Pea Gravel	250	25.0

In the fall of 1984, the Pennsylvania Transportation Institute (PTI) conducted a series of experiments involving different combinations of trucks negotiating different types of arrestor beds. Based on these experiments, a series of empirical equations were developed to predict the distance required by an arrestor bed to bring a vehicle to a rest. The equations have the form:

$$L = Av + Bv^2 + Cv^3$$

Where A, B, and C are constants depending on the grade of the arrestor bed. The PTI believed that the FHWA equation underpredicted the distance required to bring a vehicle to a rest because it did not explicitly account for planing effects. Planing effects can be defined as the phenomenon whereby when a vehicle enters an arrestor bed at high speeds, the shear strength of the aggregate is such that the vehicle will not penetrate the aggregate, thus producing very little deceleration. The PTI incorporated the effects of planing with a third order velocity term in its equation. The constants, depending on various grades are summarized in Table 4.7.

Table 4.7 Constants for the PTI equation

Grade [%]	A	B	C
0	0.6	0.021	0.00092
5	0.448	0.0149	0.000314
10	0.387	0.148	0.000205
15	0.330	0.143	0.000153
20	0.292	0.0138	0.000122
-5	2.682	-0.119	0.000661

The PTI concluded that the FHWA underpredicted the required lengths for speeds greater than 50 kph. The FHWA equation was also criticized because it only considered the rolling resistance as a constant and not a function of speed.

In 1989, Wambold et al. [41] developed a mechanistic model for predicting stopping distances of trucks in gravel arrestor beds based on seven energy absorbing mechanisms. These mechanisms include direct momentum transfer, compaction, bulldozing, side shearing, air drag, grade, and rolling resistance. The model was found to predict, with reasonable accuracy, the performance of arrestor beds with river gravel. However, the model has not been validated for speeds greater than 95 kph.

In 1992, the Arizona Department of Transportation (ADOT) conducted extensive tests on seven different arrestor beds (102 tests in total). Contrary to the PTI study, they concluded that the FHWA equation was, in fact, on the conservative side. The ADOT evaluated a modified version of the FHWA formula, and demonstrated that it was able to model an actual runaway event more accurately. The ADOT study concluded, however, that the original FHWA equation should be used, as it is more practical. Suggested changes to the FHWA equation include substituting the resistance term, R, with actual deceleration in g's of the aggregate material used, and the addition of a term in the equation to account for vehicles with different wheelbases.

It is suggested that the FHWA equation be used to calculate the required length for arrestor beds and ascension lanes due to its simplicity, and flexibility. Depending on the type of aggregate used, an actual deceleration rate can be substituted directly as the R value. The use of the PTI equations is not recommended due to its empirical nature, and inflexibility.

As a final note, when using the FHWA equation for calculating the length of an arrestor bed, the tapered depth at the beginning should not be included since it contributes little in the deceleration process.

To calculate the required length of an ascension lane, the FHWA equation may be used as well, with the R value depending on the surface material (e.g. 0.015 for compacted gravel).

4.7 Vehicle Extraction

When a runaway vehicle becomes entrapped in an arrestor bed, it is unlikely that the vehicle can drive out under its own power. It is therefore essential that a paved service lane along the arrestor bed be installed to allow tow truck access. The paved service road serves two purposes. First of all, it prevents maintenance vehicles from becoming trapped in the bedding material during routine maintenance. Secondly, it provides protection for vehicles traveling on the adjacent highway. When a runaway vehicle enters a gravel arrestor bed at speed, it causes the gravel to spray out a fair distance from the bed site. As an example, gravel was observed to be sprayed the entire width of an adjacent service lane (approximately 4 m), during testing of an arrestor bed in British Columbia conducted by the author.

An adjacent service lane, with a width of 5 m should provide adequate horizontal separation. When possible, it is ideal to locate the arrestor bed below the grade of the main roadway to provide protection to vehicles traveling on the highway during or immediately following an emergency use of the runaway lane. When grade separation is not feasible some form of screening should be installed to intercept flying gravel.

To assist tow trucks in removing entrapped vehicles, hold down tow anchors should be installed along the service road. These anchors should be placed on the outer edge of the service lane as far as possible from the bed to allow a proper leverage angle to extricate the entrapped vehicle. Furthermore, they should be installed flush with the pavement to prevent being accidentally struck by an out of control runaway truck that has veered into the service lane. The anchors should be spaced approximately 35 m apart along the service road. An additional anchor should be installed 10 m before the arrestor bed to extricate vehicles that have entered the bed only a short distance. The anchors should be designed to withstand at least 50% of the total weight of the entrapped vehicle, for example, 18,000 kg for a fully loaded semi-trailer. It should also have some type of securing handle at the top to allow attachment of winch cables.

To facilitate easier removal, the entrapped vehicle may be pulled up on some type of support (e.g., 2x6 wooden plank) so that the weight is distributed more evenly, thus greatly reducing the effort required.

If vehicles have entered only a short distance from the entry point, it may be easier to tow them out along the wheel tracks formed as they traveled into the bed.

Local tow truck companies should be consulted to determine the level of vehicle extraction capability, so that proper arrangements can be made to ensure the prompt removal of an entrapped vehicle.

For extremely heavy, multi-articulated vehicles, such as a B-train, a tow truck may not be sufficient due to the difficulty in locating a suitable attachment point on the entrapped truck that can withstand high towing forces without breaking. In these situations, a crane may be used to lift the vehicle out, or the load must be removed.

4.8 Maintenance Requirements

The effectiveness of an arrestor bed is highly dependent on the condition of the aggregate material. The arrestor bed will lose its ability to stop a runaway vehicle if it is contaminated with fines, compacted due to settling effects, or if it is saturated with water and allowed to freeze. Hence, proper drainage of the bed must be provided for and maintained, especially at locations subjected to substantial rainfall, low temperatures, snow, ice and frost. Sufficient amounts of ice, frost or ponded water may render the aggregate bed useless. Improper drainage will also lead to the accumulation of fines that will fill void spaces, and cause the aggregate bed to compact.

Drainage in an arrestor bed may be provided by installing perforated cross drain pipes of an adequate diameter spaced approximately every 30 m under the gravel bed. If the potential of oil or fuel leaks from runaway vehicles poses a serious environmental hazard, e.g., if the arrestor bed is in close proximity to a fish bearing stream, an oil water separator device should be installed at the pipe drainage outflows. Seasonal variations in the water table level should also be checked to ensure that water does not saturate the aggregate material. Paving the sides or bottom of the bed with asphaltic concrete or lining the bed with plastic liners may be helpful in this regard.

During the winter months, in areas prone to freezing temperatures or extreme freezing and thawing cycles, the arrestor beds should be inspected for ice layers. Arrestor beds will lose its ability to slow runaway vehicles if snow has refrozen and bonded the gravel together. In addition, snow accumulations can hide the approach to the arrestor bed. Snow should be cleared to delineate or indicate the presence of a runaway lane. It is important to keep the approach clear of snow because a windrow of snow across the entrance may act as a ramp and cause the truck to become airborne. If a crust of ice is found to exist, it should be broken up, for example, by using a small tractor equipped with fork attachment.

An effort should be made to keep the arrestor bed free of fines and contaminants to avoid the formation of a solid mass. When the aggregate becomes impregnated with fine material, it tends to lose its free draining capabilities and becomes compacted. The fine material fills the air spaces between the pieces of round aggregate causing the runaway vehicle to ride over the top rather than sinking in and stopping. Natural runoff should not be allowed to enter the bed, as this runoff is the primary carrier of fine soil particles. Periodic inspections should be performed to determine whether an aggregate bed needs to be screened or washed due to contamination or compaction.

Hard surfaced ascension lanes require less maintenance when compared to arrestor beds. Ascension lanes should be inspected periodically for erosion, rutting, and excessive growth of weeds and vegetation, and if necessary, appropriate mitigative action such as grading of the lane should be performed.

4.9 Back-up Attenuation Devices

Back-up attenuation devices should be installed at the end of a runaway lane when the full design length cannot be achieved due to topographical constraints, or when the consequences of a runaway vehicle exceeding the design length is severe (e.g., a steep drop-off or a nearby community exists). Typical back-up attenuation devices include gravel berms or mounds or a series of standard impact barrel attenuators.

The design of gravel mounds, as suggested by AASHTO, consists of mounds 2 to 5 feet in height with 1.5:1 slopes.

The design of standard impact barrel attenuators should follow the guidelines of their respective manufacturer. Essentially, a barrier system involves a transfer of momentum and can be summarized by the law of conservation of linear momentum:

$$M_v v_i = M_v v_f + M_b v_f$$

where:

M_v = Mass of the Vehicle [kg]
M_b = Mass of the Barrier [kg]
v_i = initial velocity [kph]
v_f = final velocity [kph]

Rearranging the above equation to solve for the final velocity in terms of initial velocity, mass of vehicle and mass of barrier, we obtain:

$$v_f = \frac{1}{\left[1 + \frac{M_b}{M_v}\right]} v_i$$

The above formula may be used to estimate the number of barrels required to bring a vehicle to a rest.

A closer evaluation of using attenuation barrels as a last resort device, however, has shown them to be inadequate. This is understandable since barrel attenuators were originally designed to stop passenger vehicles with much less mass. The following assumptions were used when evaluating the effectiveness of barrel attenuators:

- Vehicle Weight: 63,500 kg, or the maximum permissible legal GVW of an 8-axle B-train
- Initial Speed: 25 kph
- Barrel Weight: 950 kg
- Barrel Configuration: 3 across, assuming barrel diameter of 0.914 m and a spacing of 0.075 m between barrels, total width of barrel configuration would be 2.892 m, approximately 0.3 m more than maximum width of a B-train, any configuration

wider than 3 barrels would not serve any additional benefit since they will not be struck by the vehicle and could not transfer the momentum of the truck

It has been shown that the preceding momentum equation above underestimates the total deceleration capabilities of the barrels because it does not take into account bulldozing effects. It has been determined that once a final velocity is below approximately 15% of the initial impact velocity, the vehicle has essentially come to a rest.

With these assumptions in mind the following table summarizes the results:

Table 4.8 Barrel Attenuation Calculations for a B-Train

Row	M_b [kg]	V_i [kph]	V_f [kph]	Cumulative Distance [m]
1	2850	25.0	23.9	1.0
2	2850	23.9	22.9	2.0
3	2850	22.9	21.9	3.0
4	2850	21.9	21.0	4.0
5	2850	21.0	20.1	4.9
6	2850	20.1	19.2	5.9
7	2850	19.2	18.4	6.9
8	2850	18.4	17.6	7.9
9	2850	17.6	16.8	8.9
10	2850	16.8	16.1	9.9
11	2850	16.1	15.4	10.9
12	2850	15.4	14.8	11.9
...	...	...	...	...
31	2850	6.7	6.4	30.7
32	2850	6.4	6.1	31.6
33	2850	6.1	5.9	32.6
34	2850	5.9	5.6	33.6
35	2850	5.6	5.4	34.6
36	2850	5.4	5.1	35.6
37	2850	5.1	4.9	36.6
38	2850	4.9	4.7	37.6
39	2850	4.7	4.5	38.6
40	2850	4.5	4.3	39.6
41	2850	4.3	4.1	40.5
42	2850	4.1	4.0	41.5
43	2850	4.0	3.8	42.5

The calculations, as shown in the above table, indicate that a configuration consisting of 3 barrels in width and 43 rows is required to bring a B-train (63,500 kg) traveling at 25 kph to a rest at a distance of approximately 40 m. By comparison, an arrestor bed with gravel capable of providing 0.30 g of deceleration force can bring the same vehicle to a rest in almost half the distance.

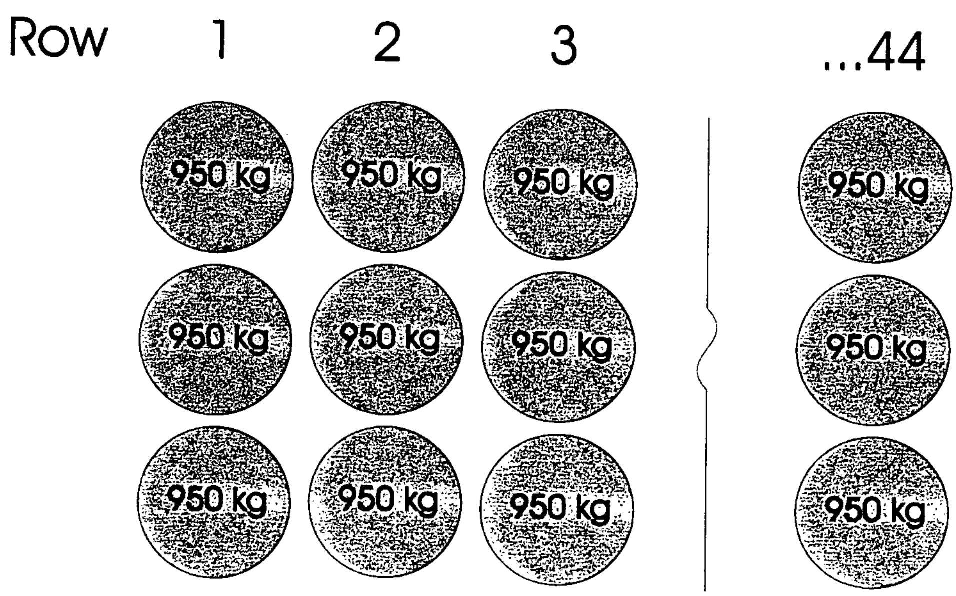

Figure 4.8 Barrel Attenuation Configuration

It would seem that placing barrels as a last resort device would have minimum benefit in terms of bringing a truck to a rest quickly. Realistically, however, after striking 10 rows of barrels, it is highly likely that accumulations from the barrel fill (gravel) would spill over and simulate an arrestor bed and contribute significantly more to the deceleration of the vehicle (through bulldozing effects) than just momentum transfer alone

It is recommended that the material used in barrel attenuation devices be filled with the same material used in the arrestor bed to avoid contamination of the bed.

Alternatively, a dragnet type system may be used. Again, the respective manufacturer should be consulted for design purposes.

Back-up attenuation devices should only be considered when the consequence of exceeding the runaway lane length is severe. For example, barrels would act as a cushioning device (before a rock face, for example). Devices such as dragnets would act as retaining devices (at the end of an ascension lane where a steep drop off follows, for example).

4.10 Approximate Costs for Runaway Lanes Facilities

The costs associated with runaway lane facilities are highly dependent on the site conditions. It depends on the length required, the grade characteristics, earthwork costs, et cetera. An arrestor bed type runaway lane facility, including a pea gravel arrestor bed, approaching and departing lanes and a service lane would cost approximately \$500,000-\$700,000. An ascension lane, if constructed with a highway project would cost approximately \$100,000-\$300,000. These costs are based on the Province of B.C. Ministry of Transportation and Highways Construction and Rehabilitation Estimating Book, March 1994.

The following is a brief list of items that should be considered when estimating costs:

- right of way or expropriation of land if required [hectares]
- clearing and grubbing of the site [hectares]
- earthwork, cut and fill [m^3]
- pea gravel for arrestor bed [m^3]
- asphalt for paved service road [tonne]
- drainage pipes [m]
- guardrails or concrete barriers [each]
- tow anchors [each]
- signing [m^2]

- delineation markers [each]
- lighting [each]
- attenuation barrels [each]

5.0 Evaluation of the FHWA GSRS for British Columbia

5.1 Grade Severity Rating Systems

One approach to reduce the likelihood of runaway vehicles is to be able to provide accurate information to heavy vehicle operators, such as the severity of the downgrade conditions they can expect to encounter. This task may be accomplished through the implementation of a grade severity rating system. In its most basic form, a grade severity rating system classifies the severity of a grade based only on the physical length and steepness of a particular segment of road. More recent grade severity rating systems, however, have gone a step further than just providing a classification system based solely on topography. For instance, the grade severity rating system developed for the Federal Highway Administration (FHWA GSRS) has the ability to provide recommended safe descent speeds based on length and steepness of a grade, the weight of a vehicle, and brake fade phenomenon. This type of system not only tells the heavy vehicle operator what conditions will be encountered on the downgrade, but provides some instruction on how to descend the grade safely through the use of a recommend safe descent speed. The following sections provide a brief chronology of events that led to the development of the current FHWA grade severity rating system.

The USBPR Grade Severity Rating System

One of the first grade severity rating systems was developed prior to 1960, and categorized downgrade severity based solely on the percent and length of the grade. The U.S. Bureau of Public Roads (BPR) developed a system that placed all downgrades into three categories (see Table 5.1).

Table 5.1 The BPR Grade Severity Rating System

Grade	Length
Greater than 3 %	Greater than 16 kilometres
Greater than 6 %	Greater than 1.6 kilometres
Greater than 10 %	Greater than 0.3 kilometres

Source: Bowman, Brian L. Grade Severity Rating System Users Manual. Washington, D.C.: Federal Highway Administration, August 1989. Report No. FHWA-IP-88-015.

Hykes

During the early 1960's, the BPR grade severity rating system was expanded by Hykes to include ten categories. Downgrades were rated from 1, the least severe, to 10, the most severe. This information was presented in a graphical form (see Figure 5.1).

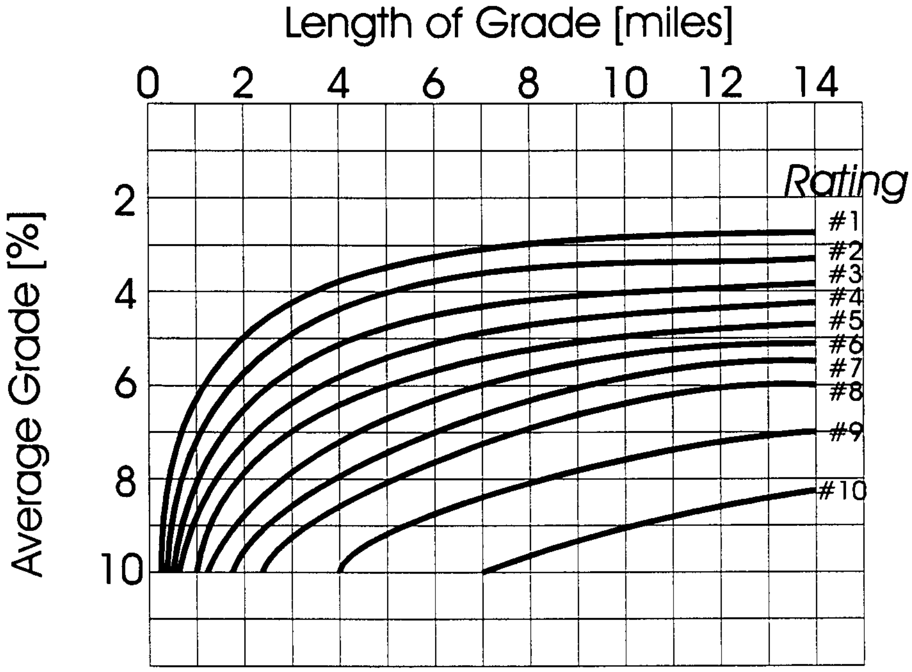

Figure 5.1 The Grade Severity System by Hykes

Lill

In 1975, Lill [8] proposed another grade severity rating system and significantly expanded previous systems through the inclusion of a representative design vehicle into the rating system. Several new concepts included rating the downgrades by their effects on a representative heavy vehicle, considering the effects of downgrade length on brake fade phenomenon, and using stopping distance criteria as a measure of braking capacity. There were limitations, however, to Lill's system: the brake fade model was empirically fit to specific test data and was not flexible enough to incorporate variables such as ambient temperature, initial temperature of the brakes, and the heat capacity and heat transfer characteristics of the brakes.

Federal Highway Administration (FHWA)

In 1979, under the sponsorship of the Federal Highway Administration, a mathematical model was developed to predict brake temperature during a descent, based on grade length and steepness, truck weight, and truck speed [8]. This model successfully correlated brake failure with high brake temperatures, and provided the foundation for the development of the current FHWA grade severity rating system.

The FHWA sponsored several additional studies [4,12,25] to further the research efforts of the initial 1979 study on brake fade phenomenon, and to develop a workable grade severity rating system.

The current FHWA GSRS, made available in 1989, consists of a computer program written in BASIC, and is accompanied by a users manual that details step by step instructions for implementation. The computer program is based on a mathematical model that takes into account the grade length and steepness, truck total weight and speed, outside ambient temperature, ambient wind conditions, and initial brake temperatures to predict brake temperatures for 5-axle vehicles not equipped with vehicle retarders. Brake temperature profiles are generated, and are used to determine recommended maximum safe descent speeds for different categories of truck weights.

The recommended maximum safe descent speed assures that brake temperatures remain below a predetermined brake temperature limit to prevent brake fade, and allows for an emergency stop at any point on the downgrade. This information may then be conveyed to heavy vehicle operators through Weight Specific Signs or WSS (see Figure 5.2). What distinguishes the FHWA GSRS from earlier systems is the capability of not only recognizing severe downgrades, but also providing useful information on how to descend the downgrade safely. In addition to providing recommended safe descent speeds, the FHWA GSRS program can be used as a tool to identify locations where brake failure is most likely to occur, thereby identifying the need for and possible locations for installation of counter measures such as runaway lanes.

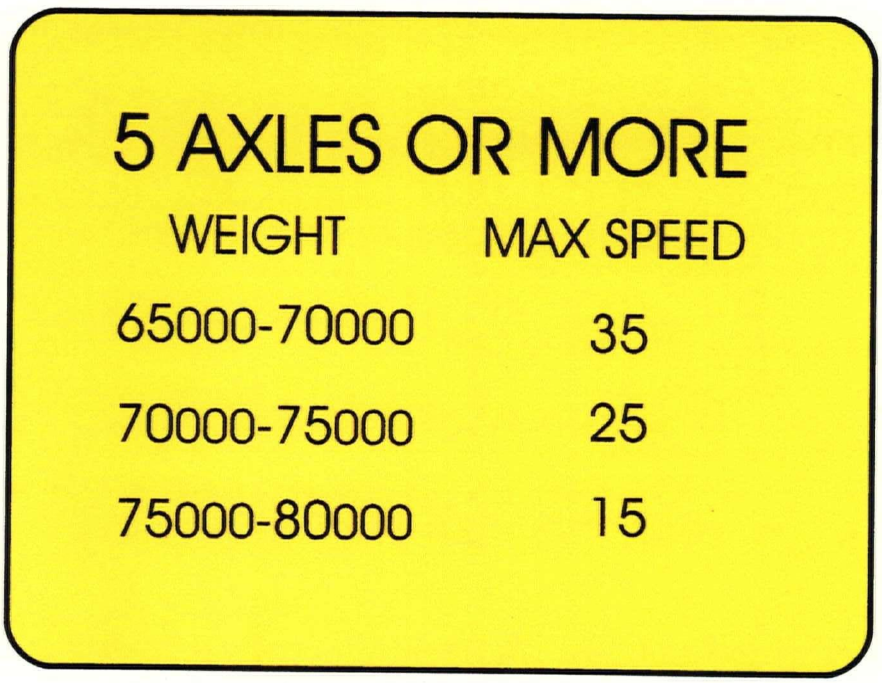

Figure 5.2 Weight Specific Sign

5.2 The FHWA GSRS for Use in British Columbia

Applications of the FHWA Grade Severity Rating System

The key advantage of the FHWA GSRS from previous efforts is the ability to advise what actions the driver should take in order to safely descend a downgrade, rather than merely describing the downgrade conditions they will encounter. This is accomplished through the implementation of Weight Specific Signs (WSS) which display a recommended safe descent speed for a particular weight class. Recommended safe descent speeds are defined as that speed from which an emergency stop at the bottom of the downgrade will not generate brake temperatures above a preselected brake temperature limit.

In addition to determining safe descent speeds, the FHWA GSRS program can be used for other applications such as establishing the need for and location of runaway lanes. This may be achieved by generating brake temperature profiles to identify where brake fade is most likely to occur. The program can be useful as an analytical tool to determine potentially hazardous grades before a truck accident occurs.

Finally, the FHWA GSRS represents the state of the art in brake fade accident prevention, and can improve the liability position of highway agencies [12].

In the development of the brake temperature model used in the FHWA GSRS computer program, a series of controlled and uncontrolled validation tests were performed [15]. In the controlled test, a single instrumented vehicle (34,246 kg 3-S2 tractor semi-trailer) was used over a variety of hills. Overall correlation between the predicted temperature and measured average brake temperature was positive and well correlated (0.85 correlation coefficient). Another test using an instrumented 2-S1-2 doubles unit tested at 35,380 kg also confirmed that the predicted temperatures from the brake model were accurate. Brake temperatures were measured by thermocouples imbedded in the brake linings during these tests.

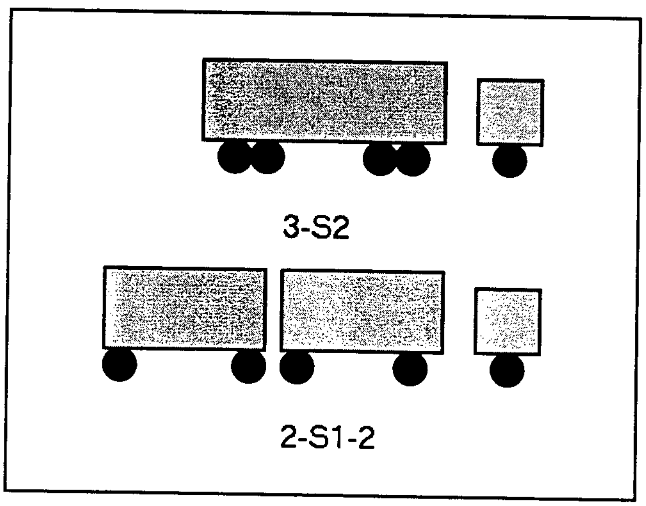

Figure 5.3 SAE Vehicle Type Designations

In the uncontrolled test, 25 randomly chosen trucks on the 'Grapevine' grade, south of Bakersfield, California were utilized. The overall correlation between the predicted temperature and measured average brake temperature was slightly lower than the controlled tests (0.81 correlation coefficient), but still highly correlated. It should be noted that there was more variability in these tests, namely different trucks types and the method of temperature measurement (the brake temperatures were measured by a hand-held portable pyrometer on each drum brake, which was observed to be consistently 17 degrees Celsius lower than the thermocouple readings). In addition, no significant differences in predicted vs. measured brake temperature for vehicles with four braking axles were found.

FHWA GSRS Modifications Due to Recent Fuel Conservation Measures in Trucks

Recent advancements to improve the fuel economy of trucks has resulted in reduced nonbraking forces that retard a truck's forward motion. Fuel conservation measures include improved aerodynamics from airfoils, streamlined tractor designs, and reduced frontal areas. The increasing use of lower profile radial tires (5 cm smaller in diameter to typical tires) has prompted another problem: the rate of kinetic energy absorption of the brakes has increased due to the faster revolution of the smaller diameter tires. Finally, improvements in truck engines has resulted in less engine friction.

The reduction in drag of new vehicles equipped with fuel conservation measures prompted a study to reevaluate the drag equations used in the development of the FHWA GSRS [15]. The results of the evaluation determined that a grade increase of 0.34% over the actual grade be used in the calculations for brake temperatures to account for fuel conservation measures. For example, if the actual grade is measured to be 8%, then the corrected grade that should be used for analysis should be equal to 8.34%.

Limitations of the FHWA GSRS for Use in British Columbia

There are several limitations of the FHWA GSRS for use in British Columbia. In British Columbia, two factors, the maximum permissible gross vehicle weight (GVW) and the number of axles, are significantly different from those used in the development of the FHWA GSRS program. The FHWA GSRS program is based on a 5-axle truck with a maximum GVW of 36,000 kg (80,000 lb), whereas in British Columbia, vehicles can attain a GVW upwards of 63,000 kg (140,000 lb), and may be comprised of up to 9 axles [37].

The FHWA GSRS program also does not have the ability take into account heavy vehicles equipped with engine retarders. It assumes that the majority of the kinetic energy dissipation is through the service brakes only.

Another limitation of the FHWA GSRS program is the inability to recognize the effects of severe horizontal curvature or sharp turns. In order for a heavy vehicle to negotiate a sharp turn without the possibility of losing stability and rolling over, the vehicle must reduce its speed. As a result, the service brakes must be applied, thereby potentially increasing their temperature. According to Hissen [27], "if sufficient leeway is left in the brake temperature limits for a complete stop, the exact position, or even the presence of sharp corners requiring deceleration is not important from the view of generating maximum thermally safe limits for truck grade descent." The concern, it seems, is not the potential for large brake temperature increases in sharp corners, and thus a loss of braking capacity reserve, but the possible discrepancy in the recommend safe descent speed assigned by the FHWA GSRS and the speed that will allow the heavy vehicle to negotiate a sharp turn safely without rolling over. For example, the FHWA GSRS may assign a recommended safe descent speed that is higher than the speed for which a heavy vehicle can safely negotiate a sharp turn.

Finally, a suitable predetermined brake temperature limit at which brake fade occurs has not yet been realized. Values in the range of 200-340 degrees Celsius have been suggested by several sources [1,8,38]. The FHWA GSRS software uses 237 degrees Celsius as the threshold temperature before brake fade occurs. Further research into this area is suggested due to the large variability of suggested values of maximum temperatures for brake fade to occur. In the interim, a threshold temperature of 237 degrees Celsius should be adequate.

Suggested Modifications to the FHWA GSRS for use in British Columbia

To address some of the limitations of the FHWA GSRS program for use in British Columbia, the following modifications are suggested.

Determining an Appropriate GVW for use in the GSRS Software

In the development of the brake temperature model in the FHWA GSRS program, the 'default design vehicle' used is a 5-axle truck with a maximum GVW of 36,000 kg (80,000 lb), and not equipped with a vehicle retarder. It is reasonable to assume that the kinetic energy that will be dissipated by a truck as it descends a downgrade will be divided equally among the braking power typically available on a 5-axle truck (i.e., 5 braking axles). In British Columbia, however, trucks can have up to 9 axles, with 8-axle vehicles being common. As a result, the number of braking axles available to retard the vehicle has increased, but not proportionally to the increase in the corresponding maximum permissible GVW. For example, a 5-axle semi-trailer (GVW 39,000 kg or 7,800 kg/axle) will be required to dissipate more kinetic energy per axle than an 8-axle B-train (GVW 63,000 kg or 7,900 kg/axle) if they travel according to the speeds suggested by the GSRS software. It would not be valid to use the GSRS software for vehicles with more than 5-axles, without some modification, because the increase in the number of braking axles has not been accounted for, and the speeds suggested may be too conservative. As braking capacity is a function of the number of axles and GVW, the concept of using a GVW/axle ratio is proposed. In order to transform a vehicle with more than 5 braking axles into an equivalent 5-axle vehicle, to be consistent with the original parameters of the brake temperature model in the FHWA GSRS program, a design GVW/axle ratio must first be chosen. It may be suitable to choose the worst case scenario or one that is most representative of the truck composition in a particular area. In British Columbia, the maximum permissible GVW for heavy vehicles is summarized in Table 5.2.

Table 5.2 Maximum Permissible GVW According to the Number of Axles

Number of Axles	Maximum GVW [kg]	Maximum GVW/axle [kg/axle]
5	52,000	10,400
6	60,500	10,083
7 or more	64,000	9,143

Source: Province of British Columbia Motor Vehicle Branch, April 1995

For example, to use the FHWA GSRS program for a vehicle with 6 braking axles, the vehicle weight used in the analysis should equal 50,415 kg or five times its GVW/axle ratio. Similarly, to estimate brake temperatures for a vehicle with 8 braking axles, the vehicle weight that should be used in the analysis should equal five times the GVW/axle ratio of an 8 axle truck. This method attempts to bypass the 5-axle limitation of the FHWA GSRS program by distributing the total GVW among the number of braking axles available on a vehicle. It should be noted that this procedure, although valid in theory, has not yet been field tested.

Determination of a Suitable Descent Speed

Once a recommended safe descent speed is generated by the GSRS software, it must be checked to assure that it does not exceed the speed limited by horizontal curvature. It has been suggested that in order to prevent a truck from losing stability and rolling over in a sharp turn, the lateral acceleration (the acceleration normal to the direction of vehicle travel) must be kept below 0.35 g [36]. Using the centripetal acceleration formula, $a = \frac{v^2}{r}$ (where a = maximum permissible lateral acceleration to prevent rollover, v = speed, and r = radius of curvature), a safe speed can be determined to negotiate a sharp turn. This speed should supersede the speed suggested by the FHWA GSRS program if it is found to be lower.

Signing Issues

It is suggested that the Weight Specific Sign as outlined in the FHWA GSRS users manual be modified to a single speed advisory sign that applies to all weight categories of heavy vehicles. The original concept of the WSS is suitable only if the commercial vehicle fleet is composed of a similar composition (i.e., 5-axles), but vary in terms of GVW. In British Columbia, however, this is not the case due to the variability in the number of axles. It was concluded that implementing WSS in B.C. would not serve its intended purpose. A sign displaying a single speed applying to all heavy vehicles has several advantages. First of all, the exclusion of different speeds for different weight classes eliminates the possibility for the driver to extrapolate a descent speed. Drivers may have a tendency to estimate a descent speed if their truck falls between a weight range on a sign. This speed that is estimated by the driver may not be valid since the recommended safe descent speeds do not vary linearly with gross vehicle weight, and thus, linear extrapolation is not possible. It has also been documented by a particular survey [45] that truck drivers tend to misjudge, and in many instances, underestimate the weight of their trucks. Eliminating the weight classification component of the sign would resolve this problem. Secondly, the majority of highways in British Columbia are two lane undivided. Since there is no opportunity to pass (with the exception of two-lane highways in B.C. which allow passing in the opposing lane in passing lane sections where AADT is less than 4000 vehicles), a single speed sign would be appropriate; specifying different speeds for different vehicles would not be practical since the slowest traveling vehicle will dictate the speed. This is contrary to the United States, where most of the highways are 4 lanes or more, thereby facilitating passing opportunities: a different range of speeds, as provided by WSS, would be effective in this type of situation. Finally, a single speed sign without weight classifications allows its effectiveness to be measured without having to physically weigh the trucks. The only variable to measure would be speed. In WSS, two variables would have to be measured, speed and GVW, with the latter be difficult to determine without a weigh scale.

Using the UMTRI Brake Temperature Model to Validate Modifications to the FHWA GSRS for Use in British Columbia

In 1988, the University of Michigan Transportation Research Institute (UMTRI) developed a brake temperature computer program [46] using heat flow equations that closely resembled those used in the development of the FHWA GSRS. The UMTRI brake temperature program, however, was not designed as a grade severity rating system, and does not have the ability to produce recommended safe descent speeds based on brake fade. Its main purpose is to predict brake temperatures reached at intervals along a specified road profile prescribed by the user. The advantages of the UMTRI brake temperature program over the FHWA GSRS program is that it is not restricted to 5 axles; the user can specify the number of axles up to a total of 13. It also has the added feature of taking account vehicles equipped with engine retarders. One minor drawback is that the UMTRI brake temperature program requires that many more vehicle parameters identified by the user when compared to the FHWA GSRS program, and therefore loses user friendliness somewhat. Nevertheless, the UMTRI brake temperature program serves as a useful benchmark to check the validity of the modifications suggested for the FHWA GSRS program for use in British Columbia.

A total of five trial runs incorporating the suggested modifications for the FHWA GSRS were performed to compare the brake temperatures predicted by UMTRI's brake temperature model and FHWA's GSRS. Vehicle input parameters were kept as similar as possible to minimize any additional sources of error. It should be noted that the brake temperatures predicted by the UMTRI program already take into account vehicles equipped with fuel conservation measures (i.e., reduced frontal areas, aerodynamic aids, etc.). Samples of the results are as follows:

The first trial involved comparing the results of the FHWA GSRS program without any modifications with that of UMTRI's brake temperature program. The design vehicle used was a 5-axle semi-trailer weighing 80,000 lb, and not equipped with a vehicle retarder. Table 5.3 summarizes the results:

Table 5.3 Average Brake Temperatures reported by FHWA and UMTRI

Length	Grade	FHWA GSRS	UMTRI	% Difference
2 miles	5%	293° F	316° F	7.3
2 miles	7%	393° F	418° F	6.0
2 miles	9%	493° F	520° F	5.2

The results of Table 5.3 show that the two programs predict similar average brake temperatures with differences no greater than 10%. It can be concluded that the FHWA GSRS is valid for 5-axles vehicles weighing up to 80,000 lb, based on UMTRI's brake temperature model.

The second trial involved modifying the FHWA GSRS to include a grade correction of 0.34% to account for modern fuel conservation measures. Table 5.4 summarizes the results:

Table 5.4 Average Brake Temperatures Reported by FHWA and UMTRI

Length	Grade	FHWA GSRS	UMTRI	% Difference
2 miles	5%	310° F	316° F	1.9
2 miles	7%	410° F	418° F	1.9
2 miles	9%	510° F	520° F	1.9

As noted previously, the UMTRI program already takes modern fuel conservation measures into account. When the FHWA GSRS program is also corrected for fuel conservation measures, the difference in predicted average brake temperatures between the two programs becomes even smaller, in the range of two percent.

The third trial involved using a fully laden 8 axle B-Train as the default vehicle (140,000 lb). As in the first trial, no corrections were made to the FHWA GSRS. Table 5.5 summarizes the results:

Table 5.5 Average Brake Temperatures Reported by FHWA and UMTRI

Length	Grade	FHWA GSRS	UMTRI	% Difference
2 miles	5%	493° F	340° F	-45.0
2 miles	7%	668° F	449° F	-48.8
2 miles	9%	844° F	557° F	-51.5

As shown in the above table, there is a large discrepancy between the FHWA and UMTRI predicted brake temperatures. These results confirm that the FHWA GSRS program is indeed dividing the GVW of an 8-axle vehicle (140,000 lb) among five braking axles only.

In trial 4, the FHWA GSRS program is modified by using the GVW/axle methodology, as outlined in the previous section, which converts an 8 axle vehicle's GVW into an equivalent 5 axle vehicle's GVW. Table 5.6 summarizes the results:

Table 5.6 Average Brake Temperatures Reported by FHWA and UMTRI

Length	Grade	FHWA GSRS	UMTRI	% Difference
2 miles	5%	340° F	318° F	6.8
2 miles	7%	449° F	427° F	4.9
2 miles	9%	557° F	537° F	3.6

When the GVW/axle methodology is used to correct the FHWA GSRS program, the results show very high agreement with differences below 10%.

Finally, the last trial combines the modifications of trial 2 and 3 to the FHWA GSRS program, namely correcting for fuel conservation measures, and using the GVW/axle methodology. The results are summarized in Table 5.7:

Table 5.7 Average Brake Temperatures Reported by FHWA and UMTRI

Length	Grade	FHWA GSRS	UMTRI	% Difference
2 miles	5%	336° F	340° F	1.2
2 miles	7%	446° F	449° F	0.67
2 miles	9%	555° F	557° F	0.36

The predicted brake temperatures by the FHWA GSRS for an 8-axle B-train compare very favourably with those temperatures predicted by the UMTRI program.

Although a limited number of trial runs were sampled, it is evident, that, in general, the FHWA GSRS program can be used in British Columbia if the suggested modifications are incorporated.

5.3 Summary

The GVW/axle methodology, rather than GVW alone should be used in the analysis for brake temperature prediction when using the FHWA GSRS program for vehicles with 5 axles or more.

To account for modern vehicles equipped with fuel conservation measures, the grade to be analyzed in FHWA GSRS program must be corrected by the addition of 0.34%.

Vehicles equipped with engine retarders will not be taken into account at this time because not all trucks are equipped with these devices. If engine retarders are taken into account, the brake temperature profiles will be substantially underestimated for trucks not equipped with such devices. More research is needed to substantiate, whether there are a sufficient number of vehicles equipped with engine retarders to warrant inclusion, and whether the operators of these vehicles are able to use them correctly. Furthermore, Fancher, et al. [23] estimates that 45 % of all runaway incidents occur with retarder equipped trucks. This implies that truck drivers with retarder equipped trucks maintain a false impression that they can proceed down the grade at a much faster speed, and indicates that they are not fully aware of their truck retarder's capabilities.

Horizontal curvature speed should always be accounted for, and will always override the recommended safe descent speed if found to be lower.

Brake fade temperature of 237 degrees Celsius is suggested as the threshold.

6.0 Determining the Need and Location of Runaway Lanes

6.1 Utilizing the Ministry Photolog Database

The first step in determining the need for runaway lane installation is to develop a list of potential sites that are probable candidates for further analysis. Currently, photolog data from the British Columbia Ministry of Transportation and Highways contains road geometry data that can be useful in determining locations that are prone to brake fade. The photolog data files contain information on distance, grade, transverse slope, side friction, curvature, roughness, elevation, and compass bearing. Only the distance, grade, and elevation columns, however, will be useful for analysis.

The photolog data files are organized according to the highway route and segment numbers. For example, the file `003W1365.DBF` corresponds to segment 1365, heading westbound on Highway 3. Each year, the photolog stores approximately 280 files, corresponding to the total number of segments that make up the numbered highways. To cover both directions of travel the number of files to be analyzed increases twofold.

The photolog files are currently stored in Dbase or *.DBF format. These database files, however, are not structured in a format that can be interpreted easily and quickly. A spreadsheet based macro was therefore developed to filter the data into a more useful format and to display the results graphically. The following section describes how this macro filter operates.

6.2 Generating User Defined Profiles

To filter the photolog data files into a format that can be easily interpreted, a spreadsheet macro was developed to display user defined grade profiles in an easy to view format. The macro's main function is to identify *continuous* segments of a certain length of road that fall within user specified grade limits (e.g. all segments with grades between -7% to -8%) and to display them on a chart.

Instructions on how to use the user defined grade limits profile macro may be found in Appendix B. As an example, Figure 6.1 illustrates a sample output from the macro. In this example, all portions of segment 1365, heading westbound on Highway 3 that contain grades between -7% and -8% are displayed. Note that the user can quickly view segments that fall within the specified grade limits. Once these grade profiles are generated, they may be cross referenced with the brake fade potential chart (see next section), to determine whether the downgrade should be analyzed further. The following scenarios may be considered when identifying sites for further analysis:

Table 6.1 Scenarios to Consider when Selecting Sites for Further Analysis

Scenario	User Defined Grade Limits	Minimum Continuous Length [km]
1	grade = -3% or less	8
2	grade = -5 % or less	3.1
3	grade = -7 % or less	1.9
4	grade = -9 % or less	1.4
5	grade < -12 %	1.0

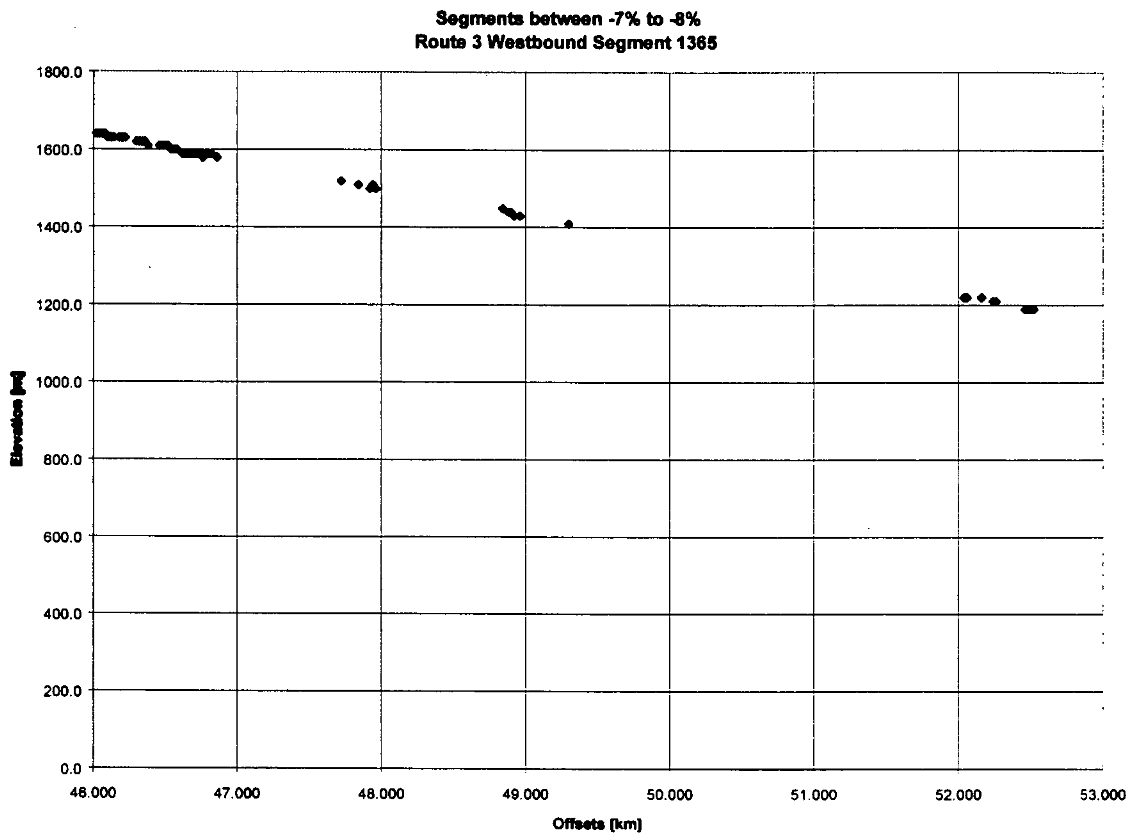

Figure 6.1 Portions of Highway 3 with Downgrades between -7% and -8%

Referring to Figure 6.1, note the continuous section of highway between offset 46 km and offset 47 km with a grade between -7% and -8%. Any continuous section with a length greater than 1 km and a corresponding steep grade, such as the example above, should be noted for further analysis because it may have the potential to cause brake fade.

6.3 Brake Fade Potential Chart Application

Once grade profiles have been generated, they may be verified for brake fade potential by using one of three ***Potential for Brake Fade on Downgrades*** charts. These charts summarize the potential for brake fade based on certain grade and length combinations. A total of three charts were generated and correspond to the distance and grade required to heat the service brakes to a point that will cause brake fade while maintaining maximum descent speeds of 60 kph, 80 kph and 120 kph respectively. The following example illustrates how these charts may be used.

Again, Highway 3, Westbound, segment 1365 will be used as an example. Figure 6.2 is the output generated by the spreadsheet macro (see Appendix B) for all portions of highway with grades less than -6% and greater than -7%.

Referring to Figure 6.2, one can see that a continuous segment is present at offset 45.7 km and 52.7 km, with an approximate length of 7 km. When cross referenced with the Potential for Brake Fade on Downgrades chart (rated for 60 kph), the potential for brake fade is in the high range, and therefore should be noted as a possible candidate for installation of runaway lanes. In other words, this location *warrants* a runaway lane based on preliminary information on grade and length.

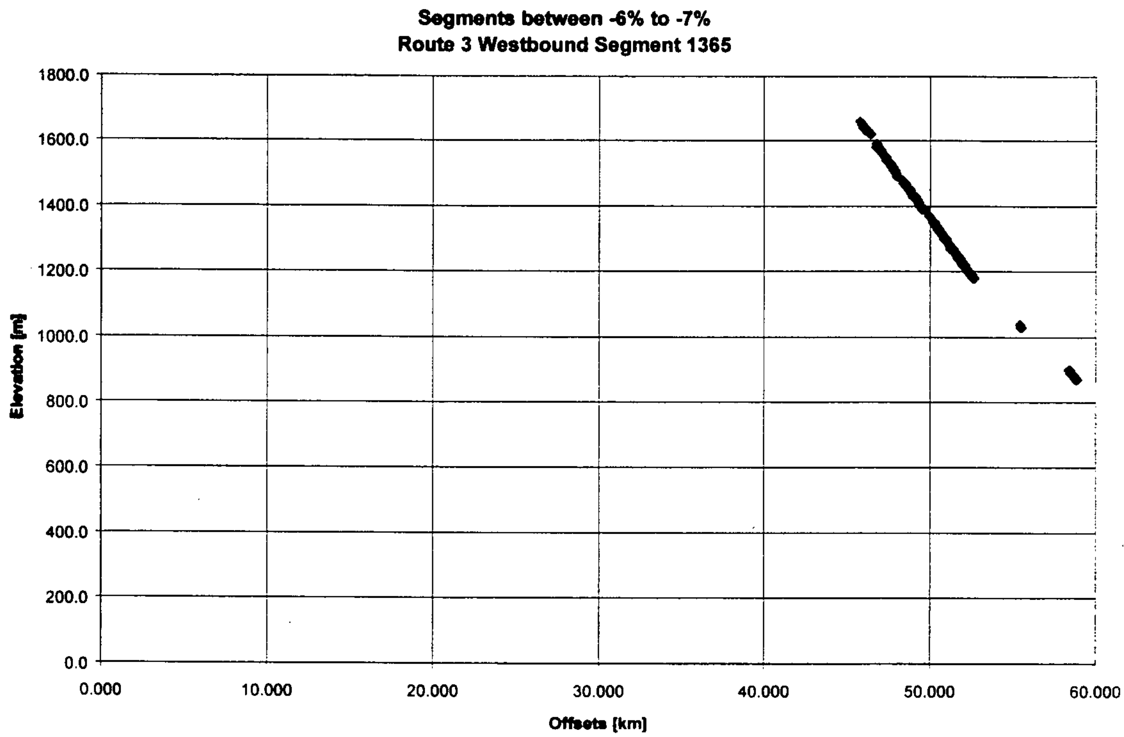

Figure 6.2 Portions of Highway 3 with Downgrades between -6% and -7%

The following figures (Figure 6.3, 6.4, and 6.5) summarize the critical combinations of grade and length that will cause brake fade and possibly a runaway situation. The charts were developed by using an eight axle B-Train as the design vehicle. The boundaries for low, high and extremely high risk for brake fade on the charts were generated using the UMTRI simplified brake temperature model. Appendix C outlines the assumptions that were made when generating these charts.

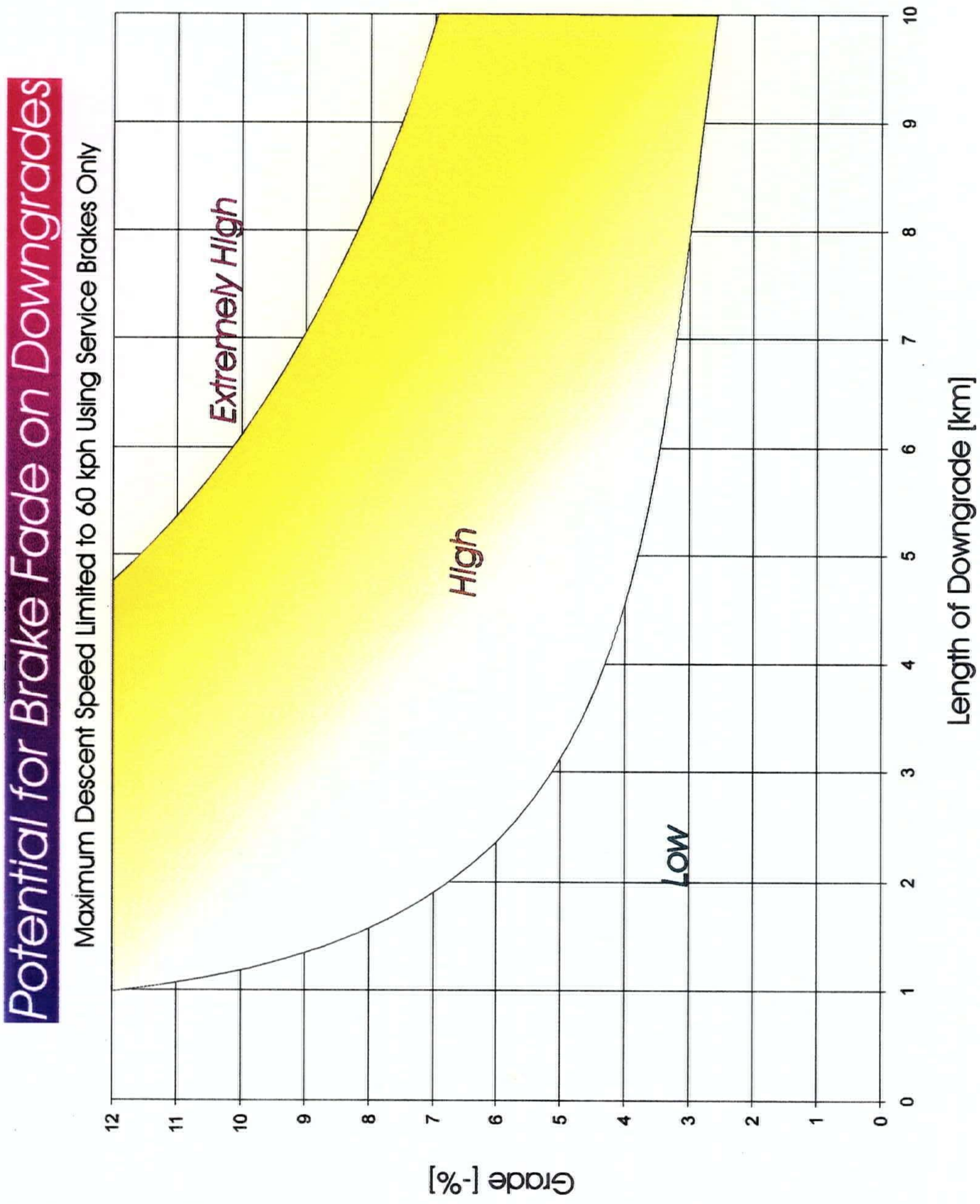

Figure 6.3 Potential for Brake Fade Based on Descent Speed of 60 kph

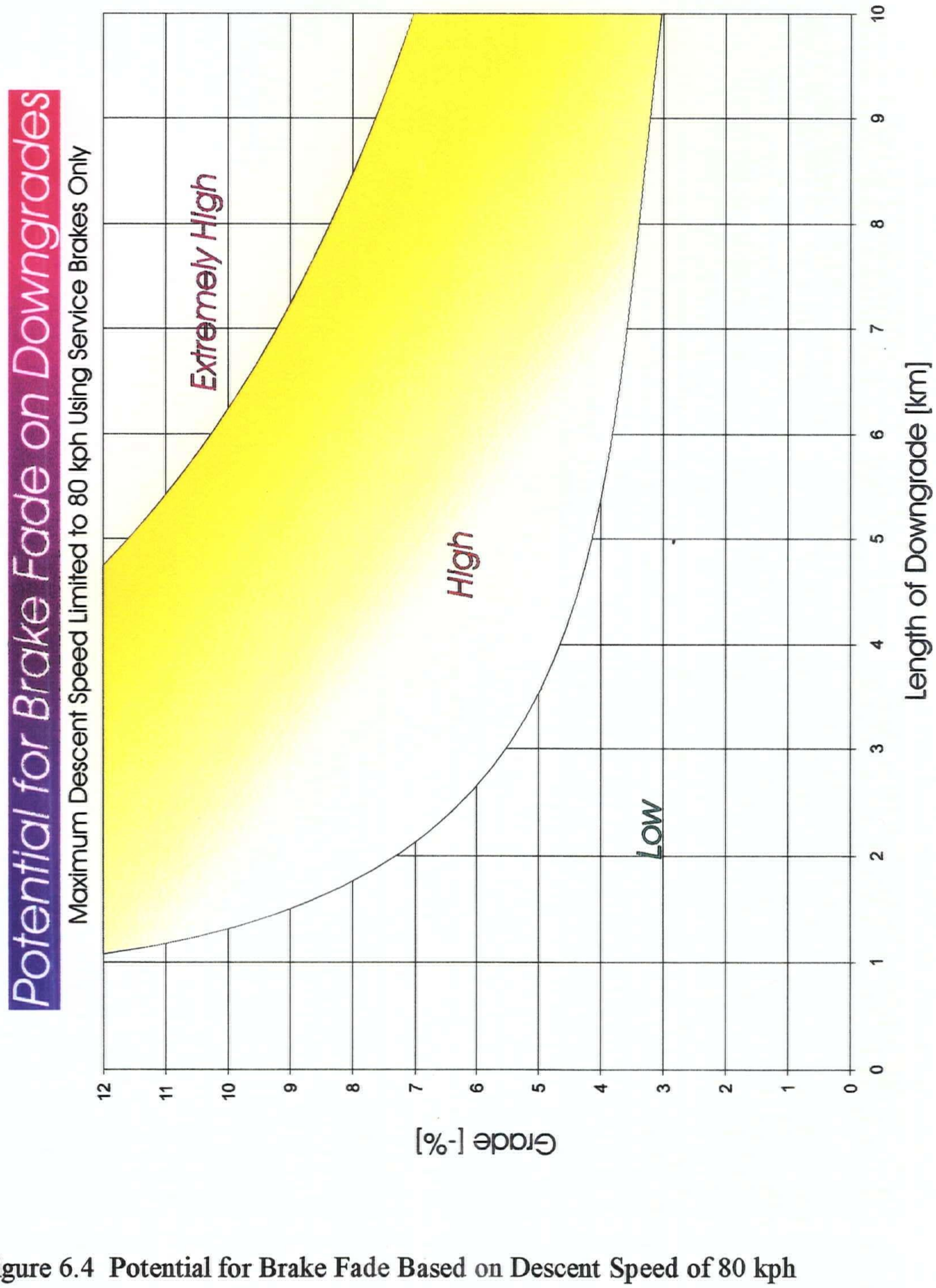

Figure 6.4 Potential for Brake Fade Based on Descent Speed of 80 kph

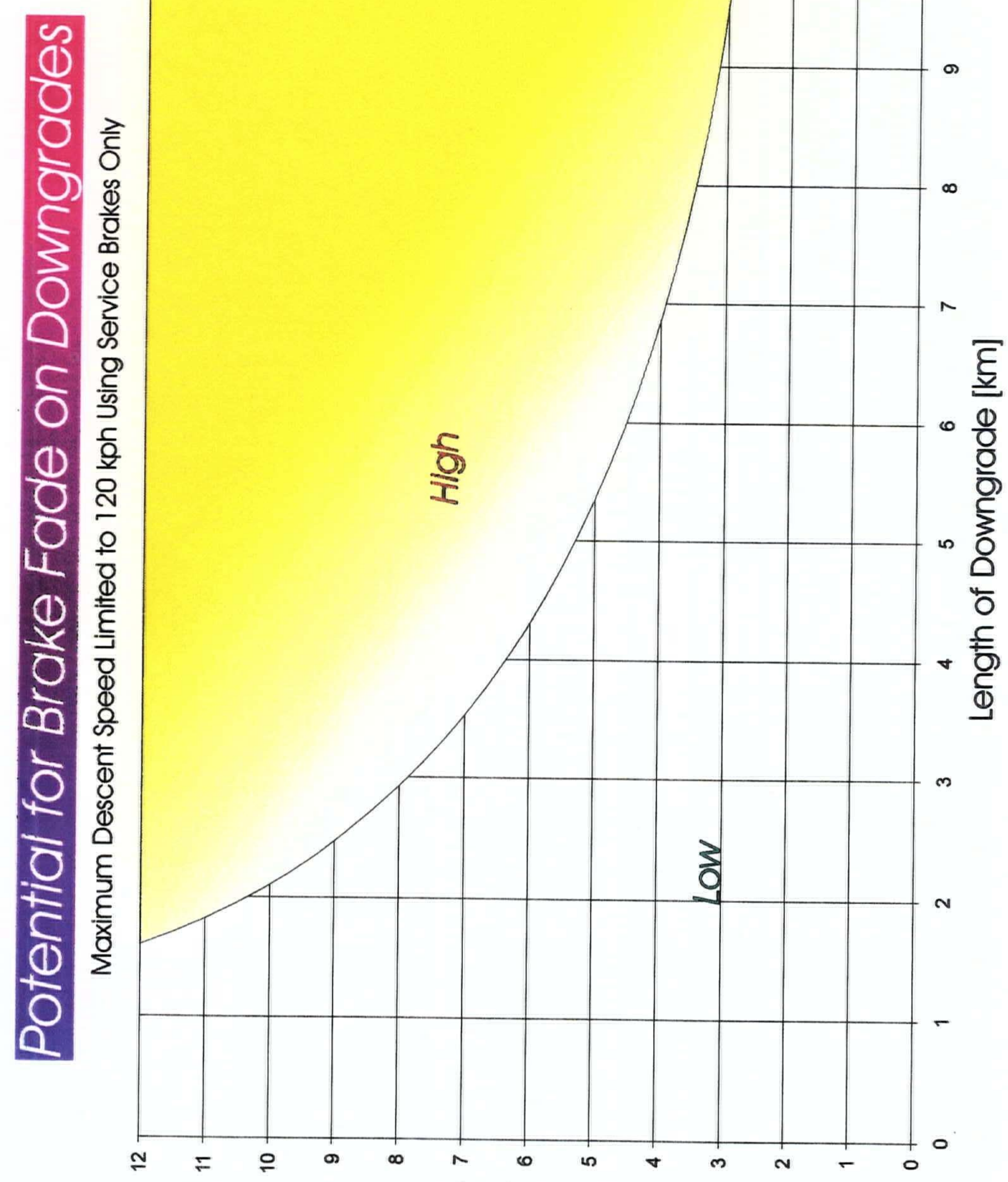

Figure 6.5 Potential for Brake Fade Based on Descent Speed of 120 kph

Figure 6.3 may be used when the downgrade in question contains curves that would limit the speed of a truck to 60 kph. In this situation, the truck is assumed to use its service brakes to maintain a speed below 60 kph to negotiate the curves on the highway, and to prevent rolling over. Figure 6.5 may be used when there are no speed limiting curves and terminal velocity of the truck is allowed to be reached without obstruction. In other words, Figure 6.5 permits more severe grade and length combinations before brake fade potential is reached and a runaway lane is warranted since the brakes are not used to limit the speed to such a great extent as in Figure 6.3. Figure 6.3 permits less severe grade and length combinations before a runaway lane is warranted because the brakes are used more frequently to keep the speed of the truck below 60 kph.

6.4 Prioritization Summary Chart

All locations that fall within the high range in the potential for brake fade charts may be plotted onto a single chart to compare relative severity.

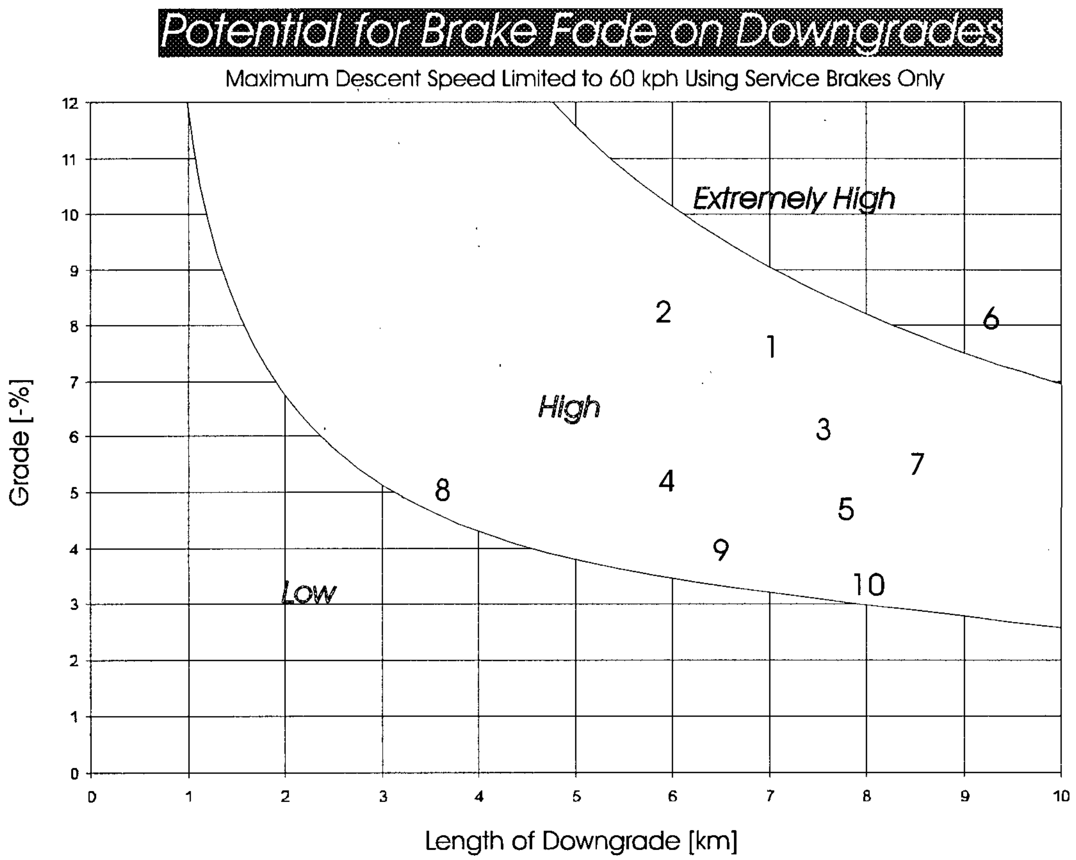

Figure 6.6 Location Prioritization Chart

Severity increases to the right and to the top of the graph. Ten hypothetical locations were plotted, and by visual inspection, the locations rated from the highest severity to the lowest severity are locations 6, 1, 2, 3, 7, 5, 4, 9, 8, 10.

Once the high risk locations have been selected, a more detailed analysis is required. More accurate data on the grade and length must be determined with field inspections. Once this data is confirmed, it may be used in either the FHWA GSRS software (with the suggested modifications), or the UMTRI simplified brake temperature model. Brake temperature profiles should be generated and plotted against the length of the downgrade to identify where the truck's brakes will reach brake fade temperature. This critical point (or points) on the brake temperature profile will identify the most ideal location to install a runaway lane.

6.5 Bayesion Accident History Analysis

The brake fade temperature based warrant described in the previous section is useful because it is able to locate and identify problematic downgrades without depending on accident history. It is able to predict where a runaway lane should be installed without having to wait for a catastrophic accident to occur. Nevertheless, we cannot ignore accident history as a basis to not only warrant runaway lanes but to recognize and prioritize existing runaway lanes facilities that need improvements.

A Bayesion analysis, taking into consideration conditional probabilities to estimate the number of accidents most likely to occur the following year was conducted on twelve randomly selected downgrades in British Columbia that contain runaway lanes. The independent variables incorporated in the Bayesion analysis include average length of downgrade, percent grade, volume, accident rate, and degree of curvature existing on the segment. The results of the Bayesion analysis were found to be more accurate than simply comparing accident frequencies alone since the Bayesion analysis is able to incorporate exposure, and road geometry in addition to accident frequency.

Table 6.2 summarizes the results of the Bayesion analysis. A detailed description of the Bayesion analysis and sample calculations may be found in Appendix D. Referring to Table 6.2, the actual number of runaway or brake failure accidents that occurred between 1991-1993 was retrieved based on police records obtained from the BC motor vehicle branch and averaged to obtained an average annual accident rate. The actual number of runaway or brake failure accidents for 1994 was also retrieved and is used for comparison purposes. Note that if we solely relied on previous accident rates to predict the number of accidents that would occur in the following year, there are several instances where previous accident rates equaled zero, but accidents occurred the following year. The Bayesion approach, however, was able to more accurately predict the likelihood of accidents occurring the following year. There was only one instance that the bayesion approach was incorrect and detrimental. It predicted that there would be less than a 50% chance that a runaway accident would occur the following year, when in fact there were two runaway accidents.

Although the Bayesion approach may be a worthwhile exercise to determine high risk locations, it requires a well maintained accident database.

Table 6.2 Actual Accident Frequencies and Bayesion Predicted Probabilities

Location	Average Accident Frequency between 1991-1993	Accident Frequency in the Following Year 1994	Bayesion Probability that more than one accident will occur in 1994 [%]
1	0	0	90.16
2	1.67	1	66
3	0	1	75.7
4	2.33	4	100
5	0	0	90.16
6	1	1	100
7	0	0	20.97
8	0	3	85.52
9	1.67	0	39.45
10	0	0	75.87
11	1	0	100
12	1	2	39.45

7.0 Future Research Requirements

A suitable predetermined brake fade temperature limit at which brake fade occurs has not yet been realized. Further research into this area is suggested due to the large variability of suggested values of maximum temperatures for brake fade to occur.

The proposed warrant has been calibrated for the worse case scenario (fully laden eight axle B-train). Vehicles equipped with engine retarders has not be taken into account. If engine retarders are taken into account, the brake temperature profiles will be substantially underestimated for trucks not equipped with vehicle retarders. More research is required to substantiate whether there are a sufficient number of vehicles equipped with vehicle retarders to warrant inclusion, and whether the operators of these vehicles are able to use them correctly.

To substantiate the Bayesion Analysis, a larger accident database must be utilized.

Finally, a study should be conducted to identify problematic downgrades in British Columbia that warrant runaway lanes, based on the methodologies presented.

8.0 Conclusions

There are several factors that contribute to a truck runaway. It has been shown, however, that the lack of driver knowledge and experience of highway terrain to be encountered is the primary cause of such accidents. In order to minimize the probability of a runaway truck situation, the driver must be well informed of the terrain that will be encountered. This may be achieved through speed advisory signs, and grade profile maps located at brake checks prior to the downgrade. Safe descent speeds may be predicted through the FHWA Grade Severity Rating System .

Runaway lane facilities should also be provided along grades which have a high potential for causing brake fade. Arrestor beds seem to be the ideal type of runaway lane for safely stopping vehicles weighing up to 63,500 kg without causing damage to the vehicle or injury to the driver. Expected deceleration rates are in the range of 0.30 g for rounded pea gravel with a depth of 450 mm. The required length of arrestor beds can be predicted using the FHWA equation.

Determining the need and location of runaway lanes can be achieved by analyzing the potential for brake fade for certain combinations of length and grade. The FHWA and UMTRI brake temperature models are useful in this regard.

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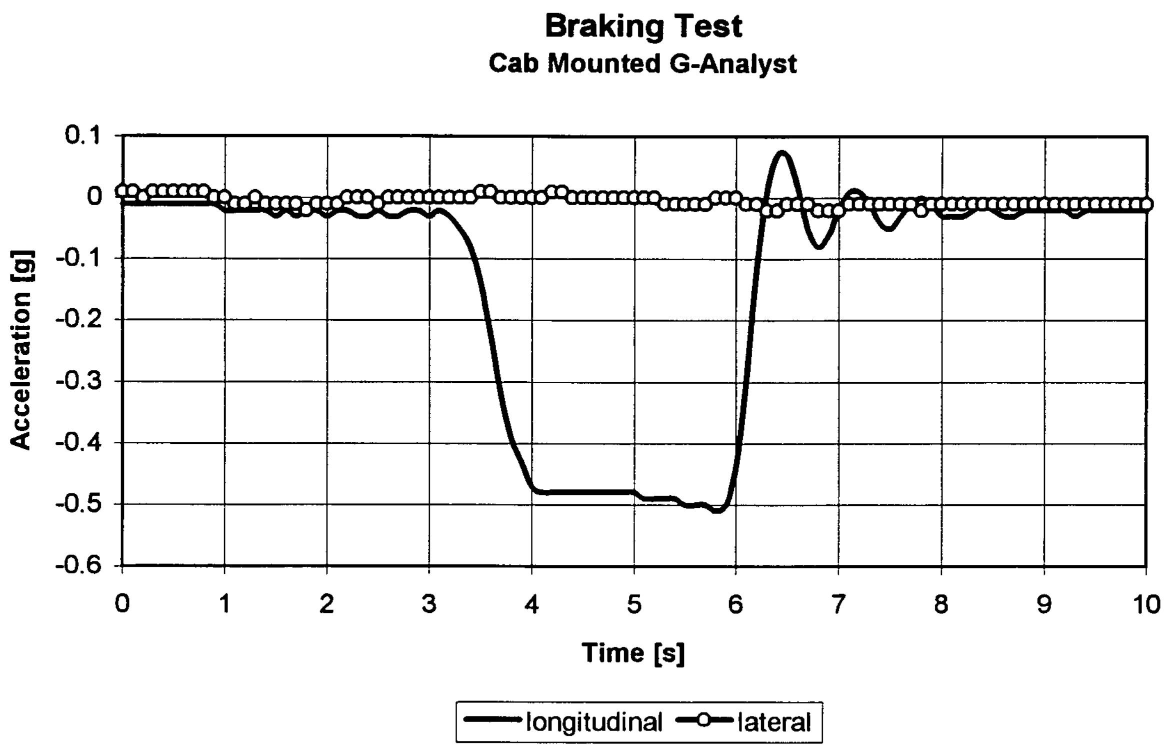
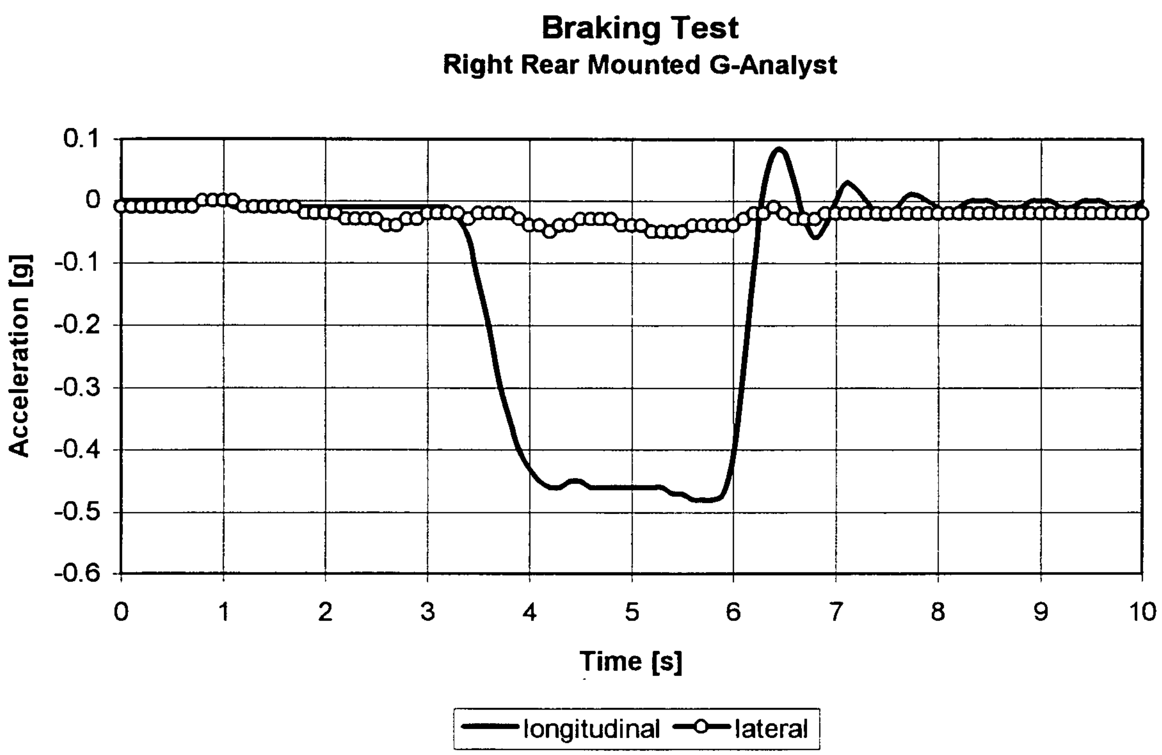
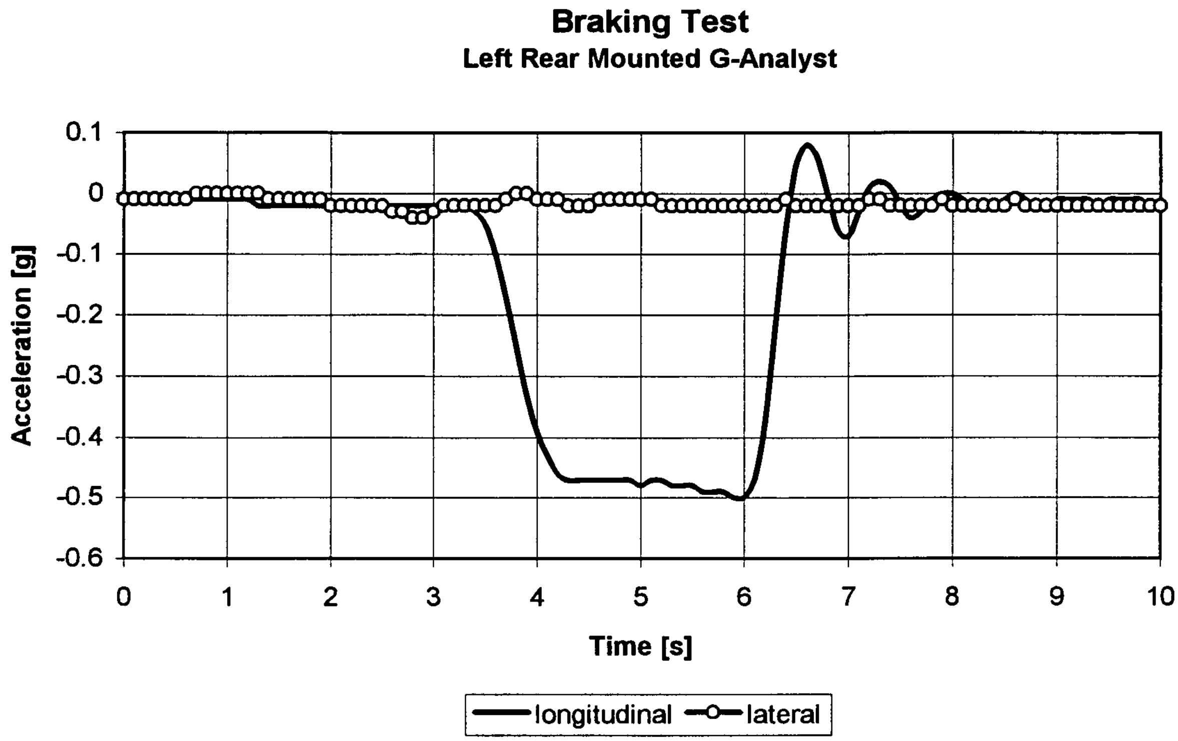
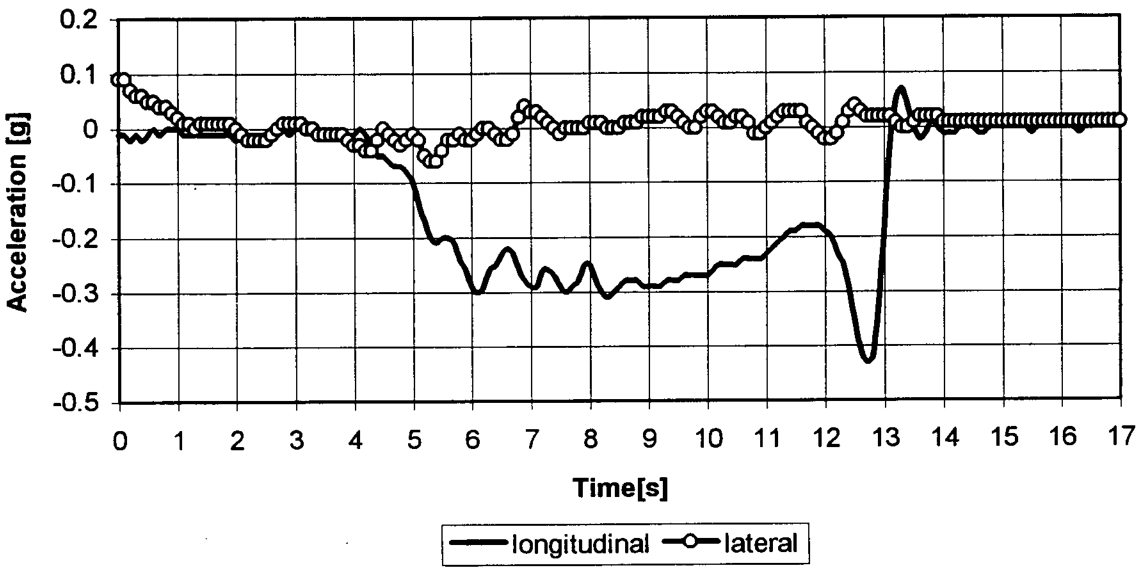

Braking Test
Cab Mounted G-Analyst
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
Acceleration [g]
0
1
2
3
4
5
6
7
8
9
10
Time [s]
longitudinal
lateral

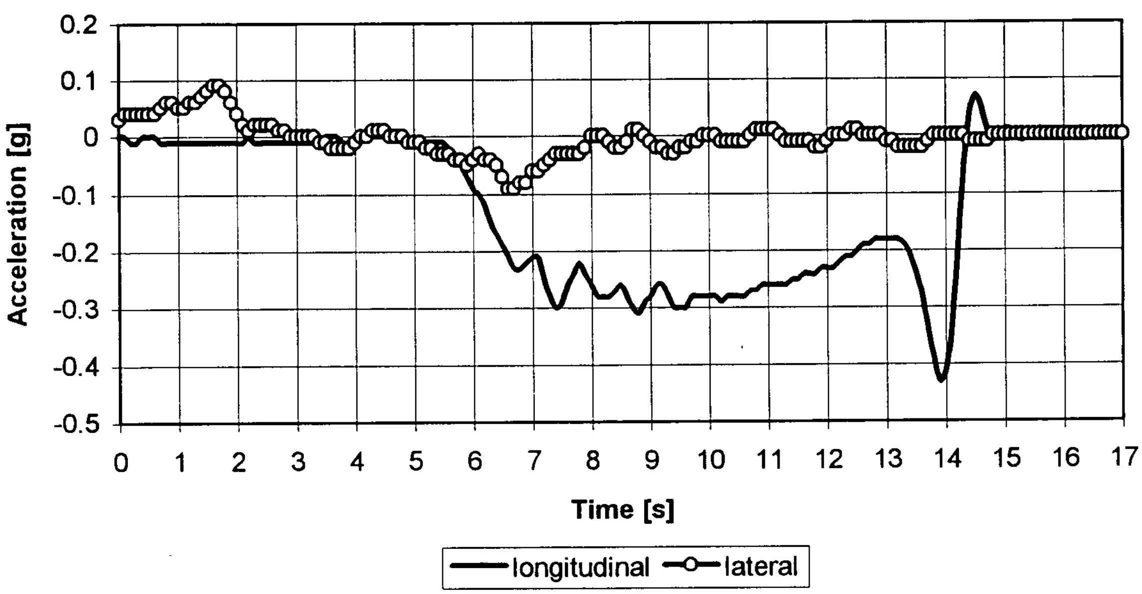

Braking Test
Right Rear Mounted G-Analyst
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
Acceleration [g]
0
1
2
3
4
5
6
7
8
9
10
Time [s]
longitudinal
lateral


Braking Test
Left Rear Mounted G-Analyst
Acceleration [g]
0.1
0
-0.1
-0.2
-0.3
-0.4
-0.5
-0.6
0
1
2
3
4
5
6
7
8
9
10
Time [s]
longitudinal
lateral

Arrestor Bed Test
Cab Mounted G-Analyst



Arrestor Bed Test
Hitch Mounted G-Analyst



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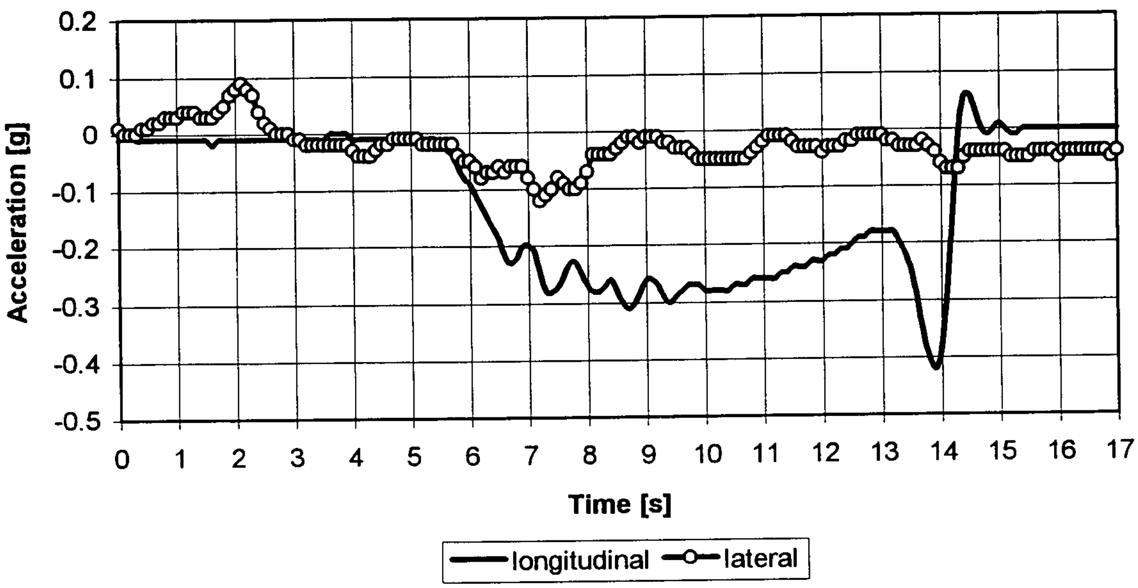
Right Rear Mounted G-Analyst



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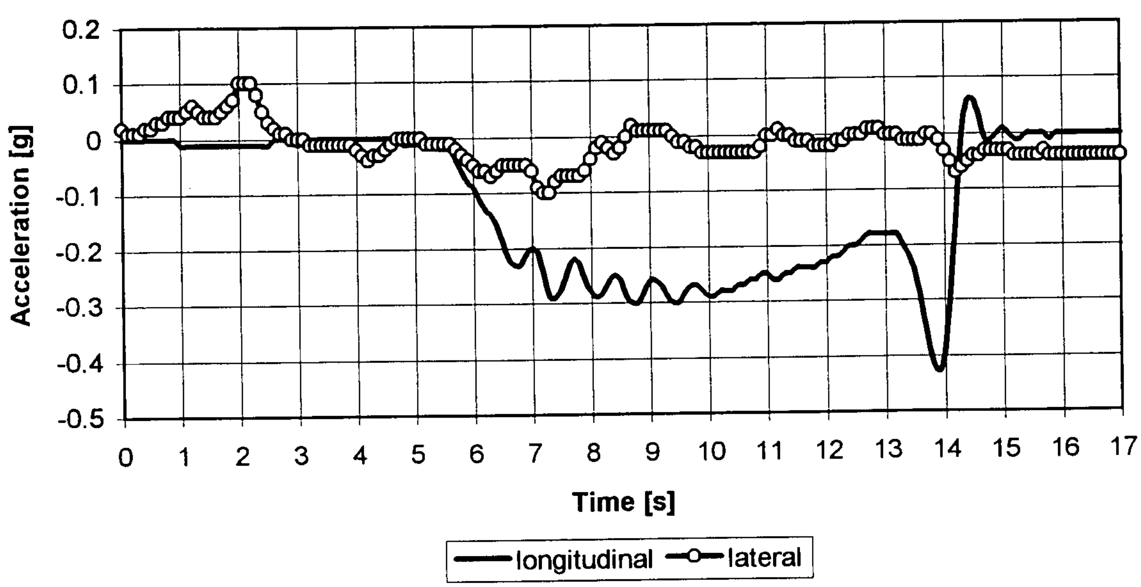
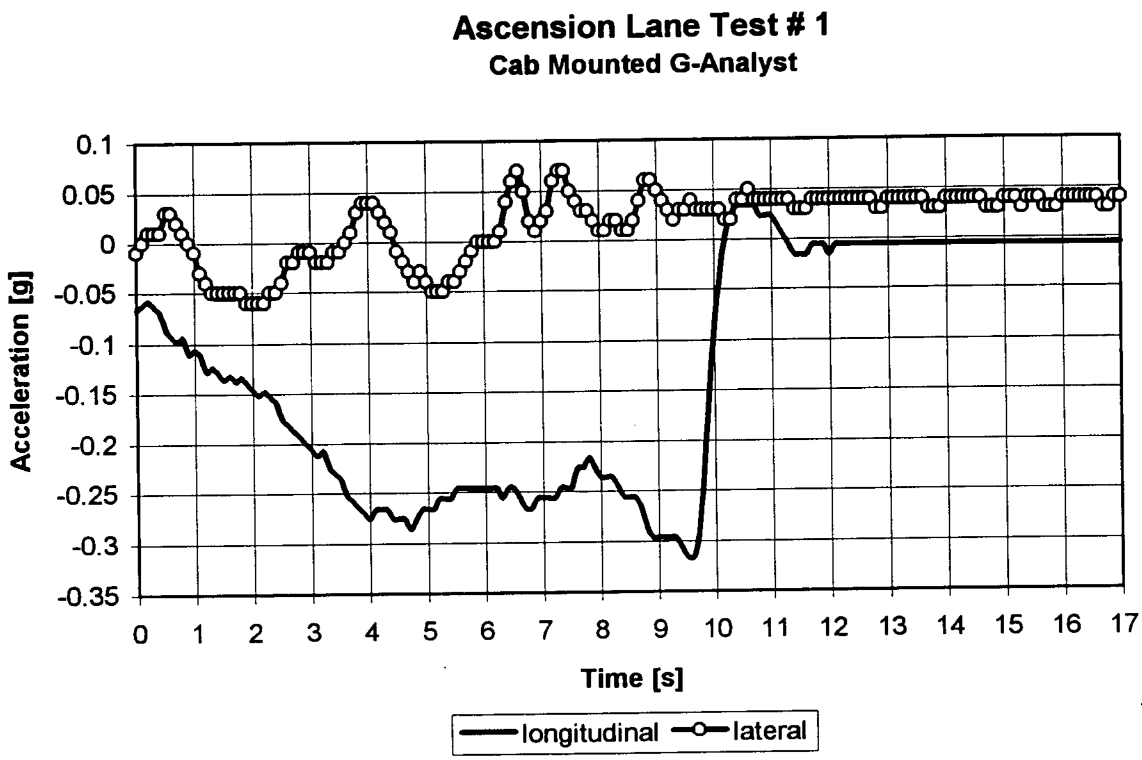
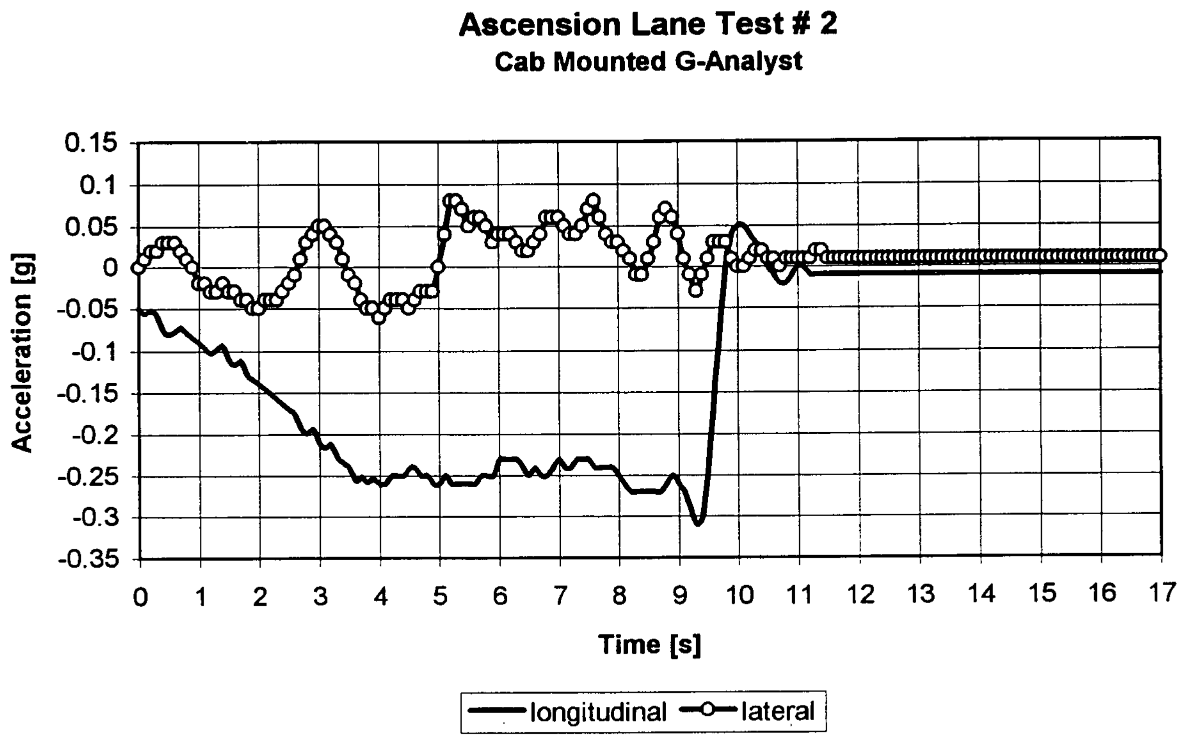
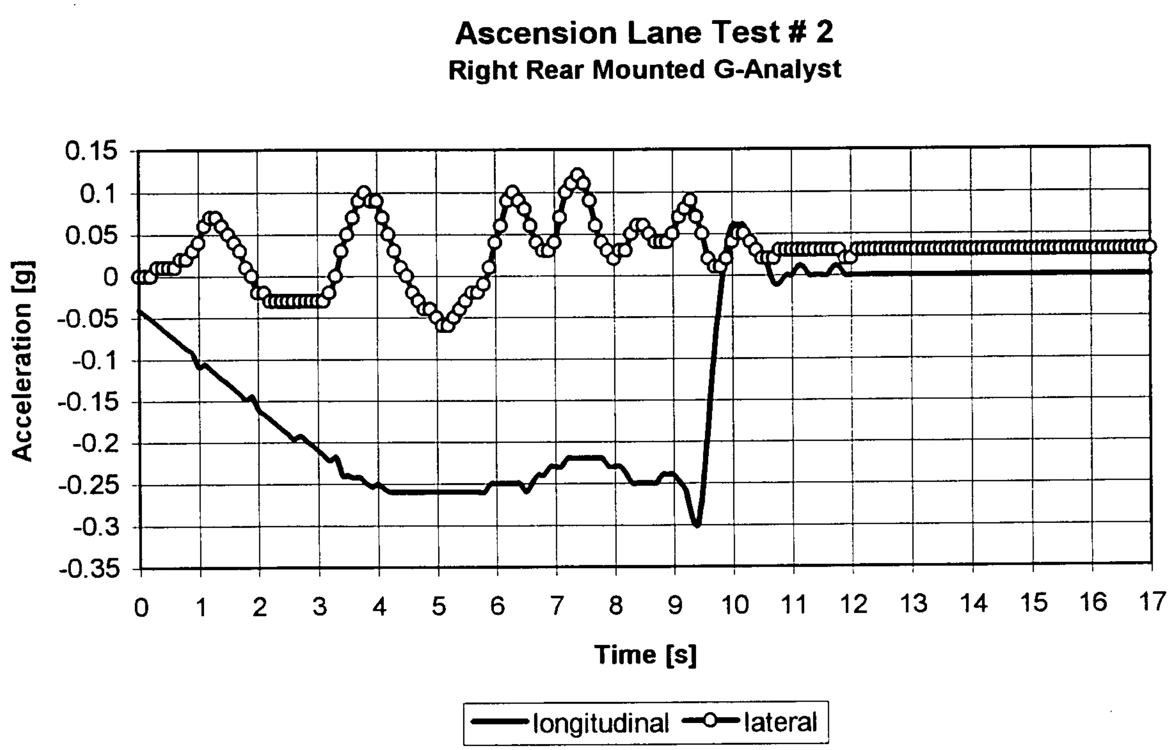
Left Rear Mounted G-Analyst



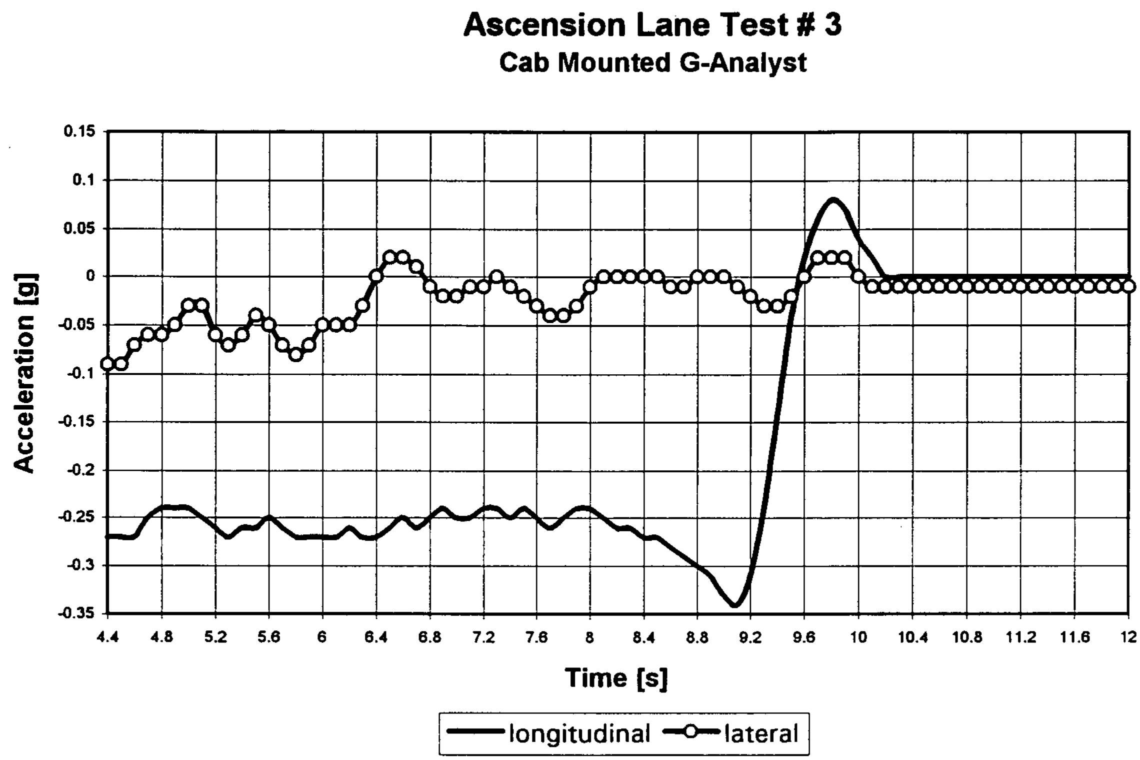

Ascension Lane Test # 1
Cab Mounted G-Analyst
Acceleration [g]
0.1
0.05
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
Time [s]
longitudinal
lateral



Ascension Lane Test # 2
Cab Mounted G-Analyst
Acceleration [g]
0.15
0.1
0.05
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17
Time [s]
longitudinal
lateral



Ascension Lane Test # 2
Right Rear Mounted G-Analyst
Acceleration [g]
0.15
0.1
0.05
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17
Time [s]
longitudinal
lateral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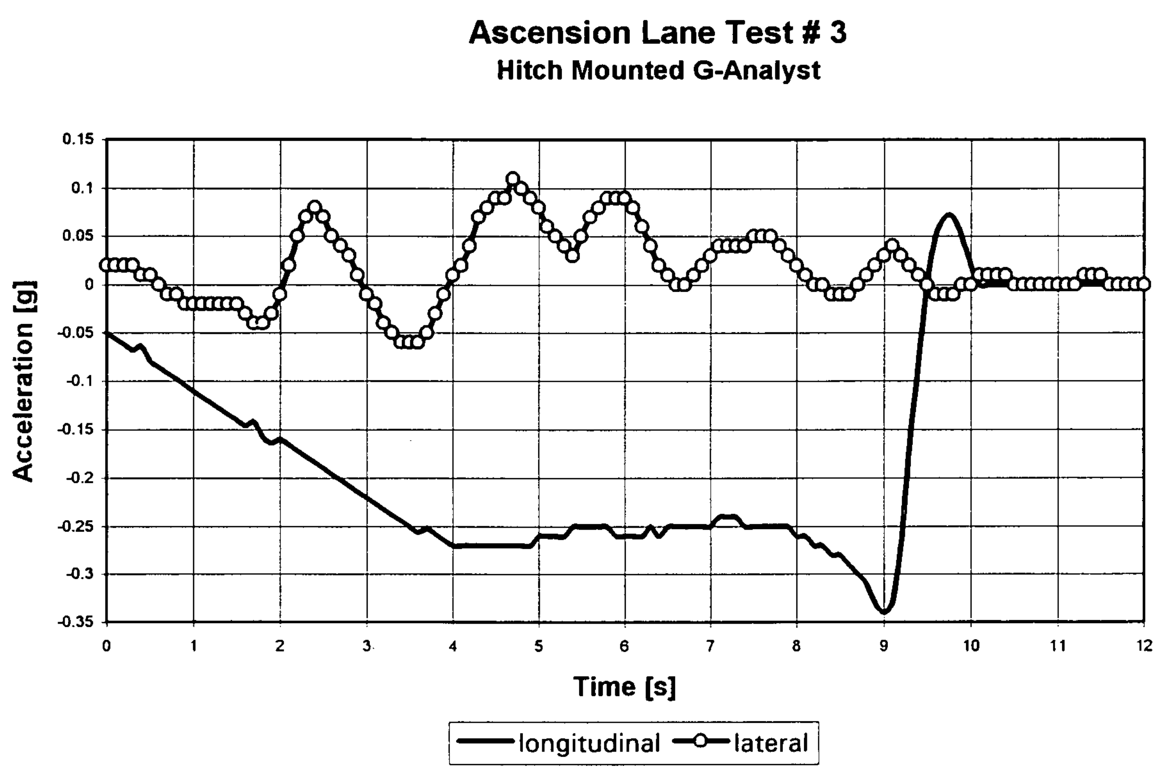
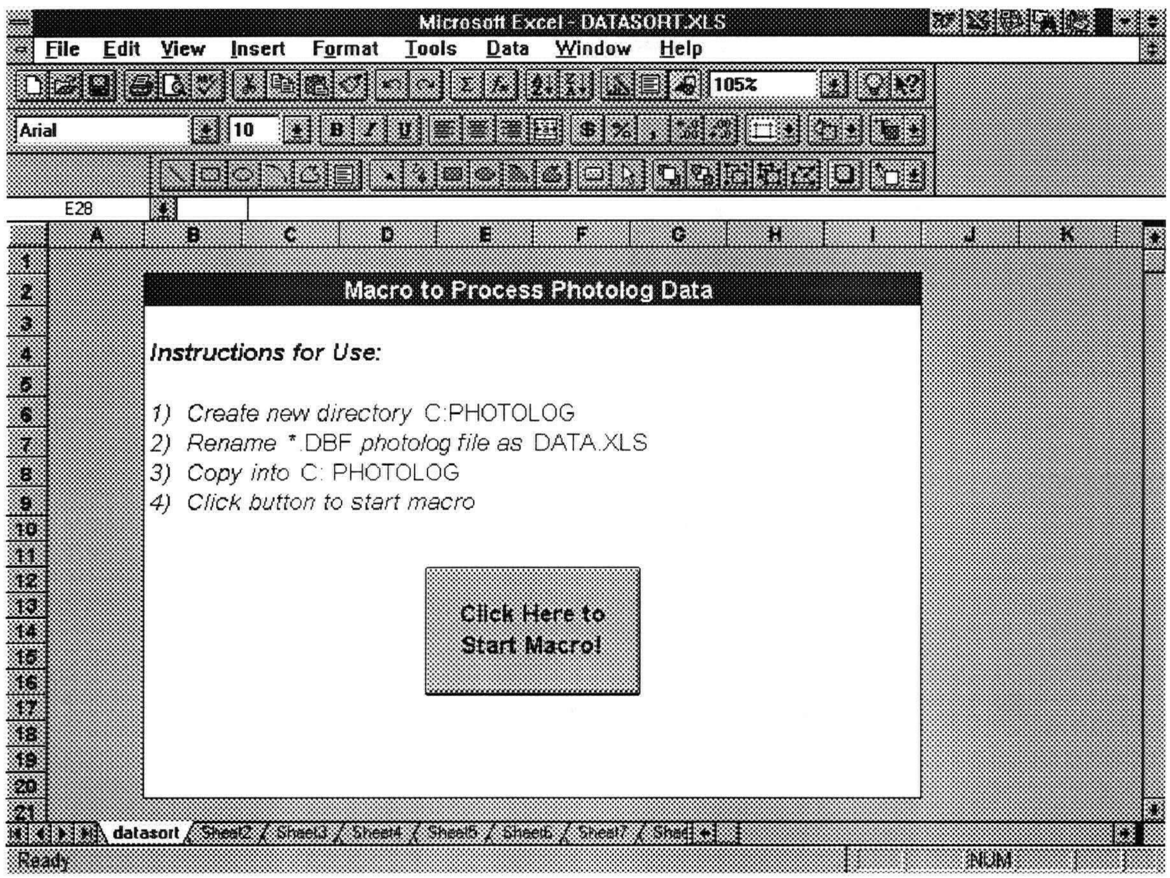
Cab Mounted G-Analyst
0.15
0.1
0.05
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
Acceleration [g]
4.4
4.8
5.2
5.6
6
6.4
6.8
7.2
7.6
8
8.4
8.8
9.2
9.6
10
10.4
10.8
11.2
11.6
12
Time [s]
longitudinal
lateral

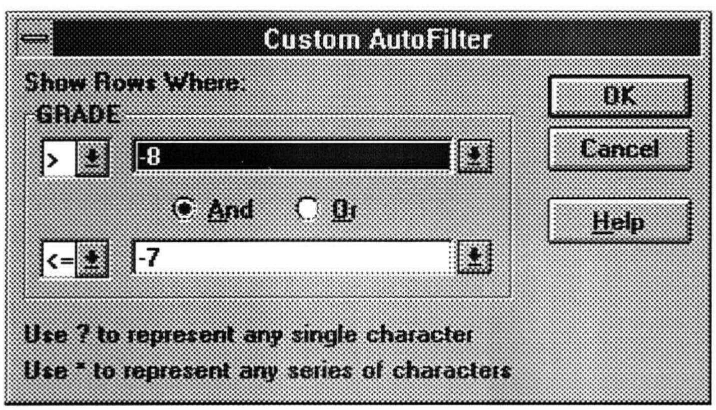

Ascension Lane Test # 3
Hitch Mounted G-Analyst
0.15
0.1
0.05
0
-0.05
-0.1
-0.15
-0.2
-0.25
-0.3
-0.35
Acceleration [g]
0
1
2
3
4
5
6
7
8
9
10
11
12
Time [s]
longitudinal
lateral

The spreadsheet based macro was developed to graphically display road segments that lie between user specified grade limits. Before the macro can be used, there are a few preliminary steps the user must perform. The first step is to obtain a photolog file which is currently stored in `*.dbf` (dbase) format. This file must be renamed `data.xls`, and saved under the directory `C:\photolog` in order for the macro to recognize and process the file. Once this has been completed the spreadsheet file `datasort.xls` may be loaded. The screen should look similar to the diagram:



To start the macro, click the button. The macro will open the file `C:\photolog\data.xls` and extract the grade, length, and altimeter columns, in addition to the route number and date for file identification purposes.

Once the macro has been fully executed a chart is generated displaying segments that lie within user specified grade limits. The default grade limits have been set between -5% to -6%. To change the default settings, select the menu button next to the grade column and select **(Custom...)**. The following Custom AutoFilter menu should now appear and the user is free to enter new grade limits. A new chart will be generated to reflect the changes.



The following is the code for the photolog macro programmed in Visual Basic.

```
Sub datasort()
    ChDir "C:\PHOTOLOG"
    Workbooks.Open Filename:="c:\photolog\DATA.XLS"
    Columns("A:A").Select
    Selection.Delete Shift:=xlToLeft
    Columns("B:B").Select
    Selection.Delete Shift:=xlToLeft
    Columns("C:C").Select
    Selection.Delete Shift:=xlToLeft
    Columns("D:D").Select
    Selection.Delete Shift:=xlToLeft
    Columns("E:L").Select
    Selection.Delete Shift:=xlToLeft
    Columns("F:I").Select
    Selection.Delete Shift:=xlToLeft
    Columns("C:C").Select
    Selection.Cut Destination:=Columns("F:F")
    Columns("D:D").Select
    Selection.Cut Destination:=Columns("C:C")
    Columns("F:F").Select
    Selection.Cut Destination:=Columns("D:D")
    Columns("A:E").Select
    Selection.AutoFilter
    Selection.Columns.AutoFit
    Columns("C:C").Select
        Selection.AutoFilter  Field:=3,  Criteria1:=">-6",
Operator:=xlAnd, _
        Criteria2:="<=-5"
    Columns("D:E").Select
    Charts.Add
                                ActiveChart.ChartWizard
Source:=Sheets("DATA").Columns("D:E"), Gallery _
            :=xlXYScatter,  Format:=1,  PlotBy:=xlColumns,
CategoryLabels _
        :=1, SeriesLabels:=1, HasLegend:=2, Title:= _
                    "User   Specified   Grade   Segments",
CategoryTitle:="Offsets [km]", _
        ValueTitle:="Elevation [m]", ExtraTitle:=""
    ActiveChart.PlotArea.Select
    With Selection.Border
```

```
        .ColorIndex = 16
        .Weight = xlThin
        .LineStyle = xlContinuous
    End With
    Selection.Interior.ColorIndex = xlNone
    With ActiveChart.Axes(xlCategory)
        .HasMajorGridlines = True
        .HasMinorGridlines = False
    End With
    With ActiveChart.Axes(xlValue)
        .HasMajorGridlines = True
        .HasMinorGridlines = False
    End With
End Sub
```

Appendix C

UMTRI Data File Description

File Name	GVW [kg]	Initial Brake Temp [°C]	Drum Weight Front [kg]	Drum Weight Rear [kg]	Final Brake Temp [°C]	Position on Downgrade [km]
gr3s60b.bkt	31750	52	29.48	47.62	102.8	10
gr3s60w.bkt	69850	103	19.16	30.95	236.1	8
gr3s80b.bkt	31750	52	29.48	47.62	231.6	10
gr3s80w.bkt	69850	103	19.16	30.95	236.8	9.5
gr3s120b.bkt	31750	52	29.48	47.62	61.7	10
gr3s120w.bkt	69850	103	19.16	30.95	168.4	10
gr5s60b.bkt	31750	52	29.48	47.62	169.0	10
gr5s60w.bkt	69850	103	19.16	30.95	236.1	3.1
gr5s80b.bkt	31750	52	29.48	47.62	161.8	10
gr5s80w.bkt	69850	103	19.16	30.95	236.0	3.4
gr5s120b.bkt	31750	52	29.48	47.62	112.0	10
gr5s120w.bkt	69850	103	19.16	30.95	236.8	5.8
gr7s60b.bkt	31750	52	29.48	47.62	235.0	10
gr7s60w.bkt	69850	103	19.16	30.95	233.0	1.9
gr7s80b.bkt	31750	52	29.48	47.62	231.0	10
gr7s80w.bkt	69850	103	19.16	30.95	237.0	2.1
gr7s120b.bkt	31750	52	29.48	47.62	167.0	10
gr7120w.bkt	69850	103	19.16	30.95	236.6	3.4
gr9s60b.bkt	31750	52	29.48	47.62	235.1	6.8
gr9s60w.bkt	69850	103	19.16	30.95	234.7	1.4
gr9s80b.bkt	31750	52	29.48	47.62	236.7	6.9
gr9s80w.bkt	69850	103	19.16	30.95	234.6	1.5
gr9s120b.bkt	31750	52	29.48	47.62	222.0	10
gr9s120w.bkt	69850	103	19.16	30.95	236.9	2.4
gr12s60b.bkt	31750	52	29.48	47.62	233.0	4.6
gr12s60w.bkt	69850	103	19.16	30.95	236.0	1.0
gr12s80b.bkt	31750	52	29.48	47.62	234.0	4.6
gr12s80w.bkt	69850	103	19.16	30.95	224.0	1.0
gr12s12b.bkt	31750	52	29.48	47.62	234.8	6.9
gr12s12w.bkt	69850	103	19.16	30.95	226.0	1.6

General Notes and Assumptions made to create the Brake Fade Potential Charts:

- The file name is broken down into four components. The first components describes the grade, e.g., gr3 represents a grade (gr) of 3 percent. The second component represents the maximum descent speed being maintained, e.g. s60 represents a maximum descent speed (s) of 60 kph. The third component describes the condition of the brakes, and GVW, e.g. b represents the best conditions, and w represents the worst conditions. Finally, the fourth component is the file extension .bkt, which is proprietary to the UMTRI brake temperature program.

- GVW was varied to represent the best and worst conditions where 31,750 kg represents 50% of the maximum permissible GVW for an 8-axle B-train and 69,850 kg represents 10% above the maximum permissible GVW.

- Initial brake temperature was changed according to previous terrain traveled. If the previous segment of road is relatively flat, an initial brake temperature of 52°C was used. If the previous segment of road is rolling, an initial brake temperature of

103°C was used. The starting temperature for flat terrain conditions of 52°C is the average temperature the brakes of a fully laden 8-axle B-Train reach after descending a 3% grade for 1 km, while trying to maintain a speed of 80 kph using service brakes only. The starting temperature for rolling terrain of 103°C is the average temperature the brakes of a fully laden 8-axle B-train reach after descending a 2% grade for 2 km, followed by a 4% grade for 2 km, while trying to maintain a speed of 80 kph using service brakes only. In both simulations, the ambient temperature of the brakes were 32.2°C.

- Drum weight of the brakes were varied to simulate ideal conditions with perfect adjustment (no reduction in typical drum weight), and poor conditions with poor adjustment (65% reduction in typical drum weight).

- A 10 km segment with varying grades were analyzed. The purpose was to find at what point on a downgrade will a temperature of 237°C be exceed. 237°C represents the point where brake fade will likely occur. In some instances the temperature of 237°C was never reached past the 10 km mark. In this case, brake fade will not likely be a problem for any segment shorter than 10 km in length. It is felt that any segment longer than 10 km should be analyzed with the UMTRI brake temperature program on a case by case basis. Any segment longer than 10 km may have large fluctuations in grade (i.e., uphill recovery zones or extremely steep sections), and assuming an average grade over a long segment may reduce the accuracy of the brake temperature prediction model significantly.

Summary Information for Bayesion Analysis

Identifier	Location	Route Name	Highway #	Segment #	Offset	Elevation Change [m]	# R.L. on seg.	Year/Disk/Side	Index	Accident Frequency 1991-1993
1	Dist.18	Barkerville Highway	26	1148	69.7-81.5	340	1	93/4/2	3485-4078	0
2	Dist.17	Chilootin-Bella Coola	20	3340	77.2-87.8	570	5	93/4/1	33807-34320	5
3	Dist.15	TransCanada	1	2050 2060 925			1	93/1/1	27905-28407	0
4	Dist.15	93 Mile to Little Fort	24	1749	80.6-96.1	840	3	93/4/1	55054-55932	7
5	Dist.14	Coquihalla	5	2025	51.9-65.2	520	2	94/2/2	7392-7940	0
6	Dist.14	Coquihalla	5	2005	67.5-90.3	840	4	94/2/2	11922-13206	3
7	Dist.9	Nancy Grn. Lk. to Rossland	3B	1335	29.4-36.7	550	1	93/2/1	23240-23649	0
8	Dist.8	Castlegar-Meadows	3	1355	20.1-35.1	650	3	93/1/2	39021-39804	0
9	Dist.8	Crowsnest	3	1305	101.7-111.3	370	1	94/1/2	28577-29149	5
10	Dist.8	Crowsnest	3	1325	126.2-142.3	770	3	93/1/2	49914-50776	0
11	Dist.8	Okanagan Connector	97C	2030	49.4-81.9	1180	2	94/6/2	20253-21665	3
12	Dist. 7	Hope-Princeton	3	1305	33.5-41.5	295	2	93/2/1	7430-7848	3

Identifier	Location	Route Name	Length [km]	Average Grade [%]	Ave. Vol. [AADT]	Curvature	Annual Ave Acc. Freq 1991-93	Accident Frequency 1994
1	Dist.18	Barkerville Highway	11.84	2.87	3875	Low	0.0	0
2	Dist.17	Chilootin-Bella Coola	10.56	5.40	1610	High	1.7	1
3	Dist.15	TransCanada	8.5	3.8	8560	Low	0.0	1
4	Dist.15	93 Mile to Little Fort	15.52	5.41	1560	Medium	2.3	4
5	Dist.14	Coquihalla	13.3	3.91	9367	Low	0.0	0
6	Dist.14	Coquihalla	22.78	3.69	8450	Low	1.0	1
7	Dist.9	Nancy Grn. Lk. to Rossland	7.3	7.53	1430	High	0.0	0
8	Dist.8	Castlegar-Meadows	15	4.33	2820	Low	0.0	3
9	Dist.8	Crowsnest	10.21	3.62	5525	High	1.7	0
10	Dist.8	Crowsnest	16.08	4.79	2479	High	0.0	0
11	Dist.8	Okanagan Connector	32.56	3.62	3750	Low	1.0	0
12	Dist. 7	Hope-Princeton	7.93	3.72	5576	High	1.0	2

											Annual Ave	Accident		
					Average		Ave. Vol.		Curvature		Acc. Freq	Frequency		
Identifier	Location	Route Name	Length [km]	X1	Grade [%]	X2	[AADT]	X3		X4	1991-93	1994	$p(y \geq 1)$	
1	Dist.18	Barkerville Highway	11.84	2	2.87	1	3875	2	Low	1	0.00	0	90.16	Incorrect
2	Dist.17	Chilootin-Bella Coola	10.56	2	5.40	2	1610	1	High	3	1.67	1	66	Correct
3	Dist.15	TransCanada	8.5	1	3.80	1	8560	2	Low	1	0.00	1	75.7	Correct
4	Dist.15	93 Mile to Little Fort	15.52	3	5.41	2	1560	1	Medium	2	2.33	4	100	Correct
5	Dist.14	Coquihalla	13.3	2	3.91	1	9367	2	Low	1	0.00	0	90.16	Incorrect
6	Dist.14	Coquihalla	22.78	3	3.69	1	8450	2	Low	1	1.00	1	100	Correct
7	Dist.9	Nancy Grn. Lk. to Rossland	7.3	1	7.53	2	1430	1	High	3	0.00	0	20.97	Correct
8	Dist.8	Castlegar-Meadows	15	1	4.33	1	2820	2	Low	1	0.00	3	82.52	Correct
9	Dist.8	Crowsnest	10.21	1	3.62	1	5525	2	High	3	1.67	0	39.45	Correct
10	Dist.8	Crowsnest	16.08	2	4.79	1	2479	2	High	3	0.00	0	75.87	Incorrect
11	Dist.8	Okanagan Connector	32.56	3	3.62	1	3750	2	Low	1	1.00	0	100	Incorrect
12	Dist.7	Hope-Princeton	7.93	1	3.72	1	5576	2	High	3	1.00	***2***	***39.45***	***Incorrect***

X1 (Length of Downgrade)	Category j	$p(X_i = j \mid y = 0)$	$p(X_i = j \mid y \geq 1)$
Less than 10 km	j=1	0.50	0.33
10 to 15 km	j=2	0.50	0.17
Greater than 15 km	j=3	0.00	0.50
X2 (Average Grade)	Category j		
-3% to -5%	j=1	0.83	0.67
Less than -5%	j=2	0.17	0.33
X3 (Average Volume)	Category j		
AADT < 2000	j=1	0.17	0.33
AADT > 2000	j=2	0.83	0.67
X4 (Curvature)	Category j		
Low	j=1	0.67	0.33
Medium	j=2	0.00	0.17
High	j=3	0.33	0.50
$p(y = 0)$	0.5		
$p(y \geq 1)$	0.5		