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Highway Planning and Design in the Qinghai–Tibet Plateau of China: A Cost–Safety Balance Perspective

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A B S T R A C T

Engineering designs for mountainous highways emphasize compliance checking to ensure safety. However, relying solely on compliance checking may lead designers to minimize costs at the expense of high risk indicators, since the overall risk level of the highway design is unknown to the designers. This paper describes a method for the simultaneous consideration of traffic safety risks and the associated cost burden related to the appropriate planning and design of a mountainous highway. The method can be carried out in four steps: First, the highway design is represented by a new parametric framework to extract the key design variables that affect not only the life-cycle cost but also the operational safety. Second, the relationship between the life-cycle cost and the operational safety risk factors is established in the cost-estimation functions. Third, a fault tree analysis (FTA) is introduced to identify the traffic risk factors from the design variables. The safety performance of the design solutions is also assessed by the generalized linear-regression model. Fourth, a theory of acceptable risk analysis is introduced to the traffic safety assessment, and a computing algorithm is proposed to solve for a cost-efficient optimal solution within the range of acceptable risk, in order to help decision-makers. This approach was applied and examined in the Sichuan–Tibet Highway engineering project, which is located in a complex area with a large elevation gradient and a wide range of mountains. The experimental results show that the proposed approach significantly improved both the safety and cost performance of the project in the study area.

1. Introduction

The Sichuan–Tibet Highway, which is located in the Qinghai–Tibet Plateau of China, is currently facing severe problems with traffic safety hazards and budget overruns. On the one hand, the Sichuan–Tibet Highway is one of the highest and most dangerous roads in the world. The lowest and highest altitudes along the way are about 500 and 5000 m, respectively. To address this topographic height difference, long vertical slopes and a large ratio of tunnels and bridges with extraordinary length have been employed in designs to cross the enormous mountainous terrain; however, these measures result in severe traffic safety hazards. Precipitous terrain and adverse weather conditions worsen the driving environment on mountainous highways and always cause a higher frequency and severity of traffic accidents than on urban highways [1]. Furthermore, the unit price of highway construction in this area is about twice that of an urban highway in neighboring areas, and the traffic mortality rate (per 1×10^5 people) is about 2–3 times higher than the national level [2,3]. Since the construction site for the Sichuan–Tibet Highway engineering project is located in a deserted mountainous area with an average altitude of over 3000 m, the manual labor and the costs for material and machinery transportation are much more expensive than in other areas.

To improve traffic safety, traditional highway planning and design often require a more conservative solution, including road-width extension and increased length of line, as a means of dealing with the large elevation difference. However, most of these improvements are costly [4]. Of course, the life-cycle cost of a highway depends on its length, geometric parameters, and infrastructure type (i.e., bridges, tunnels, etc.), which are profound latent factors in traffic safety [5–9]. Therefore, the two most significant challenges in mountainous highway planning and design are as follows: ① Dramatic topographic changes in mountainous areas make the search algorithm more likely to be trapped in a local optimum. ② The risks of cost overrun and traffic accidents are much

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more significant in mountainous highways than in urban highways; therefore, an effective optimization method is needed that can not only improve safety and reduce cost, but also strike a balance between the two.

In the existing literature, most researchers have limited the optimization objective to cost performance [10–20], since it is obvious that an optimal road design will largely reduce the total cost of a highway, including the construction, maintenance, and user costs. Some researchers have incorporated safety issues as a social cost item (i.e., accident cost) [15,17] with the aim of a total minimum objective, or have regarded safety as a design constraint [12–14,16,21–24] in order to ensure that the impact of design solutions on safety is limited to a reasonable range. Others have evaluated safety using a surrogate function (which does not involve safety-related parameters, such as slope and radius) as a validation metric for their model [25–27], or have focused on minimizing the impact of certain safety hazards [27,28]. However, such works only seek an optimal solution to minimize costs and maintain the safety performance to meet the minimum requirements of design codes.

Existing highway design codes often specify the permitted range of values for various geometrical elements, such as a maximum grade and minimum curve radius, to ensure the operating safety of users. However, conformity with the design codes does not mean that the traffic risks in a highway alignment design have been completely eliminated. First, even if the design of a highway alignment conforms to the design codes (e.g., the grade $\leq 4\%$), the accident rates can still be significantly reduced by further alignment optimization [29]. Second, in engineering practices, because most safety improvements are costly, engineers tend to adopt a minimum-cost scheme in accordance with the design codes. Such a scheme will lead to severe hazards in terms of traffic safety, and will make their methods inapplicable in mountainous areas with complex terrain. For example, if a highway alignment task is applied between two points with a large elevation gradient without considering all the other factors, the ideal solution—considering compliance checking alone—will be the shortest lane of the maximum permitted slope linking these two points (because minimum length means minimum cost). However, a better practice in engineering is to reduce the overall risk level of the highway alignment as far as possible within the budget limitation, instead of solely satisfying the constraints of the design codes. Third, the design codes do not sufficiently take into consideration the interactive effects of risk factors. For example, advanced research shows that downward grades interact with fatigue-driving behavior in highway tunnels with a monotonous and semi-closed environment, and appear to be related to traffic accidents on mountainous highways [30,31]. Therefore, a new design optimization approach based on safety risk assessment instead of on compliance checking alone is necessary, in order to ensure the risk level is within an acceptable range.

This paper describes a cost–safety optimal balance model to identify and optimize the highway planning and design variables that affect not only life-cycle cost but also operational safety. The model can not only be used for highway alignment problems, but also be extended to multi-objective design optimization for other high-risk engineering design tasks.

This work advances research on highway alignment optimization in several ways. First, it presents a new chromosomal representation form for highway planning and design, which is the basis of the cost and safety performance assessment provided here. Second, it develops an integrated assessment model by correlating the safety risk factors in design variables to the life-cycle cost items, which bridges the relationship between cost and safety. Third, it combines a fault tree analysis (FTA) with the regression method to model traffic accident causes from the perspective of highway engineering design. A generalized regression model is introduced to formulate the mathematical function between highway planning and design and the corresponding risk level (predicted crash rate) for an alignment. Fourth, a framework for an acceptable risk assessment is proposed to determine the applicability of optimal solutions in terms of traffic safety risk. Finally, a computing algorithm considering the global trend and local features of cost-efficient variations is proposed in order to strike a balance between cost and safety.

2. Formulations of the cost and safety functions

2.1. Identification of risk factors for safety analysis

Highway traffic safety is a complex objective that broadly depends on the drive–roadway–vehicle interaction [32]. Although the existing literature contains a great deal of analysis on the factors contributing to traffic accidents, few studies have been conducted from the perspective of engineering design. This section uses an FTA [33] to bridge the gap between engineering design details and traffic accidents.

Fig. 1 shows a fault tree that takes a traffic accident as the top event and the engineering design factors as the basic event, in order to develop an understanding of the logic leading from the engineering design variables to the undesired top event. It is notable that the fault tree in Fig. 1 is a newly proposed FTA diagram (designated herein as the NFTA) that was specially developed for the safety analysis of engineering designs. There are two differences between the NFTA and a traditional FTA diagram. First, the basic events in Fig. 1 are deterministic factors in a certain highway project, whereas the FTA method often uses the predetermined occurrence probabilities of basic events to deduce the probability of the top event. Second, the dependent relationships between variables are probabilistic rather than deterministic [34]. For example, even though "driver error" is an important factor leading to traffic accidents, the presence of driver error will not always result in a traffic accident. In this, the NFTA differs from the traditional FTA assumption.

The NFTA in Fig. 1 shows that traffic accidents mainly result from driver errors and vehicle-operating problems caused by poor road design.

Driver error refers to difficult situations in which poor highway design may have a negative impact on the performance of drivers. For example, super-long tunnels and a dim driving environment are important factors in driver fatigue, which prolongs the reaction time of drivers and often leads to accidents. The coupling effects of high altitude for a mountainous highway crossing a plateau and poor ventilation in a tunnel may cause a driver to become anoxic, resulting in a traffic accident.

Vehicle-operating problems refer to dangerous events in which poor highway alignment design increases the difficulty of vehicle operation and the probability of vehicle failure, even when drivers have sufficient experience and vehicles are in a good state of operation. Previous research indicates that design consistency plays a vital role in providing conformance between practical operation of a vehicle and driver expectancy [35]. A good design in terms of road consistency eliminates unexpected and abrupt changes, and thus effectively reduces the probability of an accident [27]. Design consistency can be classified into three main categories: vehicle stability, operating speed consistency on a single element, and operating speed consistency on successive elements. *Vehicle stability* analyzes whether the side frictions provided at a curve are enough to prevent a vehicle from skidding out.

Operating speed refers to the actual speed selected by drivers under free-flow conditions, which is always determined by the highway alignment geometrics. In addition to the above design

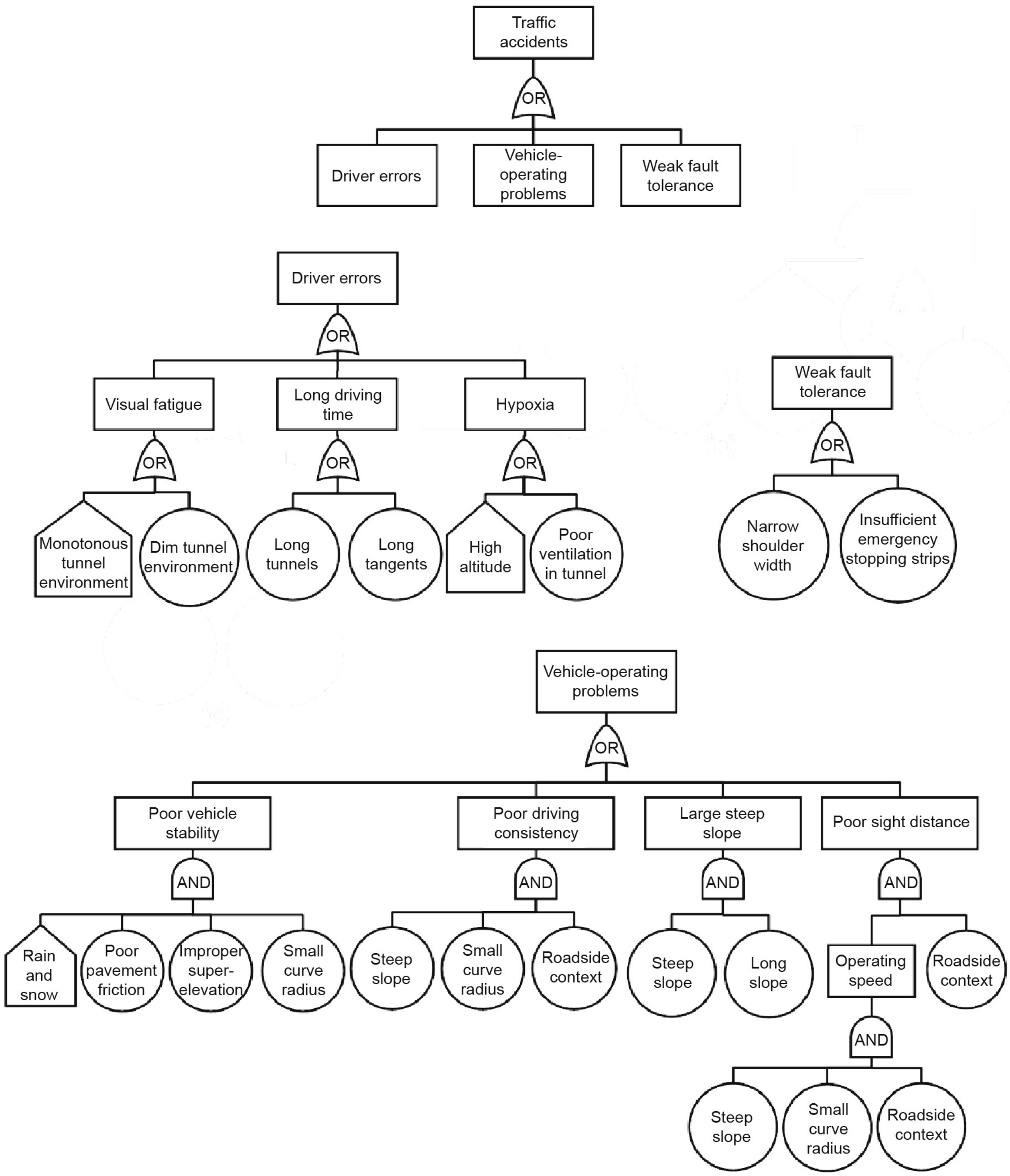


Fig. 1. NFTA of highway traffic accidents.

consistency criteria, a large longitudinal slope and poor sight distance also bring great challenges to vehicle control and may lead to an unexpected crash.

Given that drivers' mistakes and vehicle breakdowns are sometimes unavoidable, incorporating fault-tolerance facilities into a highway (e.g., road shoulders and emergency stopping strips) can effectively reduce the traffic accident rate.

Although the NFTA in Fig. 1 provides insight into factors in a highway-engineering project that can lead to traffic accidents, the inference approach of the traditional FTA method is inapplicable in this study, due to the major differences between the NFTA and an FTA. To solve this problem, the NFTA diagram was mapped onto a generalized linear-regression model in order to compute the traffic accident probability for a highway segment, given the engineering design information. The mapping algorithm is described as follows: First, the minimal cut sets [36], which refer to the minimal, necessary, and sufficient conditions for the occurrence of traffic accidents, are identified in the NFTA. Next, the minimal cut sets are converted to the explanatory variables in the regression model. In particular, the AND gate in NFTA will combine the information of the lower events to compute the state of upper events. Table 1 provides the risk factors deduced from the NFTA diagram, which will be used as the explanatory variables in the regression model.

The risk factors listed in Table 1 are important parameters that are associated with the planning and design of mountainous highways. It is worth noting that these risk factors not only affect road traffic safety, but also affect the life-cycle cost of highways.

Table 1
Risk factors list for highway traffic accidents.

Risk factors	Corresponding engineering design variables	Design variables relation with	
		Construction cost	Maintenance cost
Dim tunnel environment	Lighting design in tunnel	√	√
Long tunnels	Tunnel length	√	√
Long tangents	Tangent length	√	√
Poor ventilation in tunnel	Ventilation design in tunnel	√	√
Poor vehicle stability	Pavement friction, superelevation, curve radius	√	×
Poor driving consistency	Slope grade, curve radius, and roadside context	√	×
Large steep slope	Slope grade and slope length	√	×
Poor sight distance	Slope grade, curve radius, and roadside context	√	×
Narrow shoulder width	Road shoulder width	√	×
Insufficient emergency stopping strips	Distance between emergency stopping strips	√	×

2.2. Decision variables for preliminary highway design

Now that traffic safety risk factors have been identified in the section above, a representative framework for highway planning and design is developed in this section. Both the risk factors and the parameters related to life-cycle cost analysis can be extracted from this framework.

Let $S(x_S, y_S, z_S)$ and $E(x_E, y_E, z_E)$ be the start and end points of a highway design. The highway alignment is segmented into small homogeneous sections by inserting the points of intersection $P_i = (x_{pi}, y_{pi}, z_{pi}, R_{pi})$.

As shown in Fig. 2, an alignment is segmented into curves and tangents that are represented as $\ddot{E} = (C, T)$, where C and T are specified in Eqs. (2) and (4). Curves are represented by C,

$$C = [C_1, C_2, \ldots, C_n] \tag{1}$$

which is comprised of a set of curved segments C_i:

$$C_i = \left[x_{TC,u}, y_{TC,u}, z_{TC,u}, x_{CT,u}, y_{CT,u}, z_{CT,u}, Lv_u, Gv_u\right], \ i = 1, 2, \ldots, n \tag{2}$$

where the coordinates of transition points $(x_{TC,u}, y_{TC,u}, z_{TC,u})$ and $(x_{CT,u}, y_{CT,u}, z_{CT,u})$ can be calculated by geometric constraints [37],

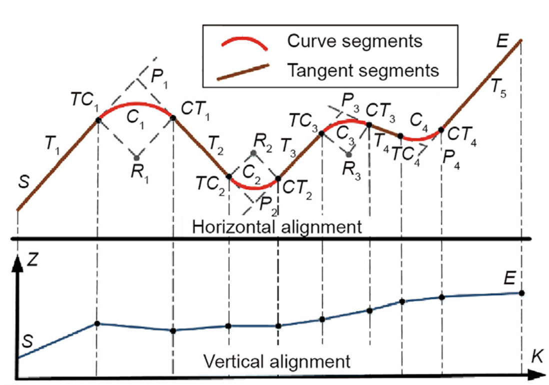


Fig. 2. Correspondence of horizontal alignment and vertical alignment.

given the horizontal coordinates of the intersection points P_{i-1}, P_i and P_{i+1}, as well as the horizontal curve radius R_{Pi}. Lv_u denotes the local variables for a particular segment, and Gv_u denotes the unchanged global variables along the whole alignment. For example, the superelevation of each section is different, but the shoulder width of each section is the same. Tangents are represented by T in the same way as curves are represented by C.

$$T = [T_1, T_2, \ldots, T_{n+1}] \tag{3}$$

which is comprised of a set of curved segments T_i:

$$\begin{cases} T_i = \left[x_s, y_s, z_s, x_{TC,2}, y_{TC,2}, z_{TC,2}, Lv_u, Gv_u\right], \ i = 1 \\ T_i = \left[x_{CT,u-1}, y_{CT,u-1}, z_{CT,u-1}, x_{TC,u}, y_{TC,u}, z_{TC,u}, Lv_u, Gv_u\right], \ i = 2, \ldots, n \\ T_i = \left[x_{CT,n}, y_{CT,n}, z_{CT,n}, x_E, y_E, z_E, Lv_u, Gv_u\right], \ i = n+1 \end{cases} \tag{4}$$

Note that the number of tangent segments is equal to the number of curved segments plus one.

Table 2 lists the design variables other than the three-dimensional (3D) coordinates of the transition points in Eqs. (3) and (5). It should be noted that some are defined as discrete variables, since finite states are given in these variables (e.g., the lighting and ventilation design in a tunnel). Given the information from Eqs. (3) and (5), the parameters required for cost and safety performance assessment can be calculated.

2.3. Estimation of life-cycle cost

Life-cycle cost f_{co} is evaluated in the form of the annual average cost, as shown in Eq. (5). Let c_f be the capital recovery factor, which is usually set at 0.065 in China.

$$f_{co}\left(\ddot{E}\right) = c_f C_C\left(\ddot{E}\right) + C_M\left(\ddot{E}\right) \tag{5}$$

where

$$C_C\left(\ddot{E}\right) = K_{TC}L_T + K_{BC}L_B + C_R + K_{FC}(L_T + L_B + L_R) \tag{6}$$

$$C_R = C_{GF} + C_{GC} + K_{PA}L_{PA} \tag{7}$$

$$C_M\left(\ddot{E}\right) = C_{TM} + C_B = K_M(L_T + L_B + L_R) + K_O L_T \tag{8}$$

Given the alignment route information $\ddot{E}$ and Geographic Information System (GIS) data, various highway structures (bridges, embankments, tunnels, or deep cuts) are automatically determined according to their life-cycle cost, as shown in Fig. 3.

Table 2
Design variables list.

Design variables	Symbols	Description	Category
Curve radius	R_i	Curve radius (m)	Local variables
Superelevation	e	Cross slope (%)	Local variables
Lighting design in tunnel	Li	Tunnel lighting reflectors only = 0, general lighting system only = 1, enhanced lighting system + general lighting system = 2	Global variables
Ventilation design in tunnel	Ve	Natural ventilation = 0, mechanical ventilation = 1	Global variables
Pavement material	Pa	Concrete = 0, asphalt = 1	Global variables
Road shoulder width	Sh	Shoulder width: 1.5 m = 0, 2.5 m = 1, 3.0 m = 2	Global variables
Distance between emergency stopping strips	Em	Distance: 500–800 m = 0, 300–500 m = 1	Global variables

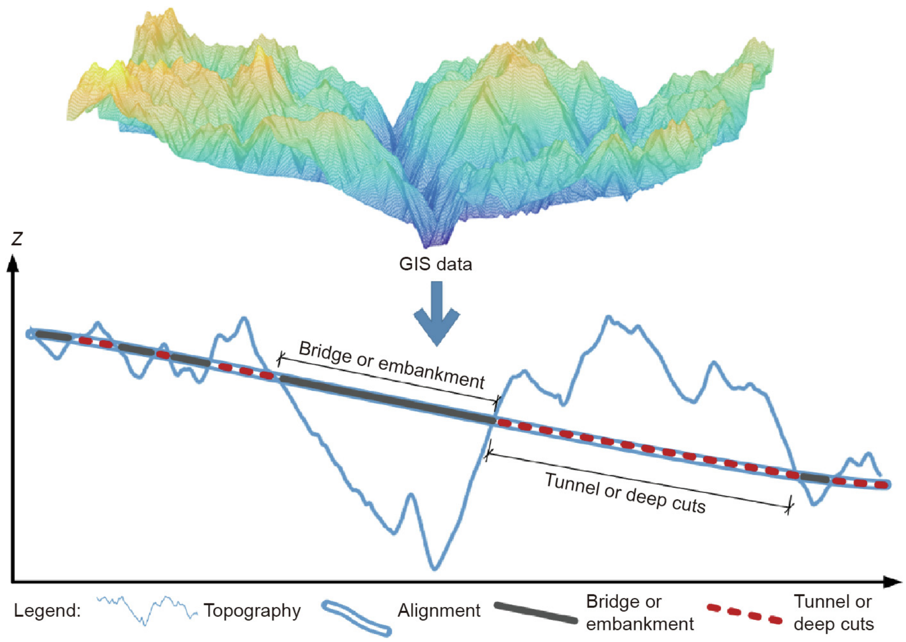


Fig. 3. Automatic layout of various highway structures. If altitude Z(Alignment) > Z(Topography), chose bridge or embankment by comparing construction cost: for Cc(Bridge) < Cc(Embankment), chose bridge, otherwise chose embankment. If altitude Z(Alignment) < Z(Topography), chose tunnel or deep cuts by comparing construction cost: for Cc(Tunnel) < Cc(Deep cuts), chose tunnel, otherwise chose deep cuts..

For example, if the maximum life-cycle cost of a tunnel is lower than that of the deep cuts in a certain segment, a tunnel is selected as the ideal highway structure in this section, and vice versa. In this way, the lengths of various highway structures are determined. Let $L_{\rm T}$ and $L_{\rm B}$ be the length of tunnels and bridges, and let the unit prices of construction be defined as $K_{\rm TC}$ and $K_{\rm BC}$, respectively. The construction cost $C_{\rm C}$ can be calculated as the sum of the related costs of the tunnel, bridge, road, and ancillary facilities, as shown in Eq. (6). The construction cost of road, $C_{\rm R}$, equals the sum of the cost of the ground-fill $C_{\rm GF}$ and of the cut, $C_{\rm GC}$, and pavement construction, $K_{\rm PA}L_{\rm PA}$, the detailed formulation of which can be found in Ref. [37], and is not covered in this paper. In addition to the main structure construction, the total construction cost includes the cost of ancillary facility procurement and installation along the whole highway; the unit price is defined as $K_{\rm FC}$. The costs for highway maintenance $C_{\rm M}$ consist of two parts: daily maintenance, and the operating expenses of the facilities and equipment. The unit prices for these two parts are defined as $K_{\rm M}$ and $K_{\rm O}$, respectively. Since hardly any rescue or electrical equipment is required for bridge and road operations, the operating expenses for bridges and roads are negligible compared with those for tunnels.

Note that most unit prices, including $K_{\rm TC}$, $K_{\rm BC}$, $K_{\rm PA}$, and $K_{\rm O}$, are not fixed values, and depend on the state of the design variables in Table 2. Eqs. (9)–(12) show the formulations for the unit prices used in this model. $K_{X,0}$ represents the base price of K_X ($K_X = [K_{\rm TC}, K_{\rm BC}, K_{\rm PA}, K_{\rm O}]$) when all the design variables v ($v = [Li, Ve, Pa, Sh, Em]$) in Table 2 have a value of 0, while $K'_{X,v}$ represents the additional cost compared with $K_{X,0}$ when the design variables v have a non-zero value.

$$K_{\rm TC} = K_{\rm TC,0} + K'_{{\rm TC},Li} + K'_{{\rm TC},Ve} + K'_{{\rm TC},Pa} + K'_{{\rm TC},Sh} + K'_{{\rm TC},Em} \tag{9}$$

$$K_{\rm BC} = K_{\rm BC,0} + K'_{{\rm BC},Pa} + K'_{{\rm BC},Sh} + K'_{{\rm BC},Em} \tag{10}$$

$$K_{\rm PA} = K_{\rm PA,0} + K'_{{\rm PA},Pa} + K'_{{\rm PA},Sh} + K'_{{\rm PA},Em} \tag{11}$$

$$K_{\rm O} = K_{\rm O,0} + K'_{{\rm O},Li} + K'_{{\rm O},Ve} \tag{12}$$

Detailed values for $K_{X,0}$ and $K'_{X,v}$ will be specified in the case study.

2.4. Estimation of traffic accident rate

To estimate the crash frequency for a highway segment, traffic accident information from the same region as the construction project must be collected from the traffic administration ministry. With the highway geometrical data, traffic data, and crash data extracted from the traffic accidents report, a multivariate regression analysis can be performed to model the mathematic function between the crash rate and the related risk factors [38,39]. This paper employs a widely used metric called the safety performance function (SPF), which was first proposed in the Highway Safety Manual [40] by the American Association of State Highways and Transportation Officials (ASSHTO) to evaluate the traffic risk level of a proposed alignment. The SPF is a surrogate function [41,42] that uses known information for a road segment to predict its traffic safety in terms of accident frequency; it has been widely used in several highway design tools, such as the Interactive Highway Safety Design Model [43], SafetyAnalyst [44], and Crash Modification Factors Clearinghouse [45]. However, all of these tools are designed for safety assessment in rural/urban highways, and thus assume a driving environment that is significantly different from the environment of a mountainous highway with a large proportion of tunnel segments. Therefore, the SPF $f_{\rm sa}$ is defined by combining all the information from segments located in a mountainous driving environment, as in Eq. (13).

$$f_{\rm sa}\left(\ddot{E}\right) = \frac{\sum_{i\subset\Lambda}(L_i\ddot{e}_i)}{\sum_{i\subset\Lambda}L_i} \tag{13}$$

where L_i is the length of segment i (a curve segment or tangent segment); and $\ddot{e}_i$ is the SPF of the ith segment, which uses a generalized linear-regression model to establish the mathematical relationship between the explanatory variables and the annual crash frequency [5]:

$$\ddot{e}_i = E(Y_i) = e^{X_{i'}\dot{a}} = e^{\alpha_0+\sum_{j=1}^{n}\alpha_j X_{i,j}} \tag{14}$$

where Y_i represents the predicted annual crash frequency of segment i, $X_i = (1, X_{i,1}, X_{i,2}, \ldots, X_{i,n})$ denotes the explanatory variables of segment i extracted from the NFTA diagram (see Fig. 1); and $\dot{a} = (\alpha_0, \alpha_1, \ldots, \alpha_n)$ denotes the corresponding coefficients, which are described in Table 3.

In general, it is assumed that Y_i follows a negative binomial distribution [46] with a variance of $\mathrm{Var}(Y_i) = \ddot{e}_i + \ddot{e}_i^2 \hat{e}$. The fundamental negative binomial regression model for a segment i is written as follows:

$$\Pr(Y_i = y_i | \ddot{e}_i, \hat{e}) = \frac{\Gamma(y_i + \hat{e})}{\Gamma(\hat{e})\Gamma(y_i + 1)} \left(\frac{\hat{e}}{\hat{e} + \ddot{e}_i}\right)^{\hat{e}} \left(\frac{\ddot{e}_i}{\hat{e} + \ddot{e}_i}\right)^{y_i} \tag{15}$$

where Γ is the gamma function and $\hat{e}$ is the dispersion parameter of the negative binomial distribution.

The regression coefficients $\dot{a}$ are estimated using the method of maximum likelihood by the GENMOD procedure in Statistical Analysis System (SAS) software [47]. Note that various statistical tests, including goodness-of-fit test, significance test, and residual analysis, should be adopted before the employment of the SPF model in practical applications [48].

3. Cost–safety optimization and balance for highway planning and design

3.1. Acceptable level of risk examination

Let R^n be the decision space defined on an n-dimension Euclidean vector space. Since each candidate highway planning and design set can be represented by $\ddot{E} = (C, T)$, it can be converted into a chromosome $\ddot{E} = (P, Lv, Gv)$. $P = (P_1, \ldots, P_k)$ contains the 3D coordinates and curve radius information for k intersection points. $Lv = (Lv_1, \ldots, Lv_{2k+1})$ contains the local variables information for the k curve segments and the $k+1$ tangent segments. Gv includes the global variables information for all the segments.

A bi-objective optimization problem for the highway planning and design in terms of cost and safety can be formulated as follows:

$$\min_{\ddot{E} \in \boldsymbol{R}^n} \left[f_{\mathrm{co}}\left(\ddot{E}\right), f_{\mathrm{sa}}\left(\ddot{E}\right)\right] \tag{16}$$

To find the best candidates for the decision-maker, the Non-Dominated Sorting Genetic Algorithm II (NSGA-II) is employed [49]. Every individual in the population requires two entities in order to be computed: non-domination rank and crowding distance. Based on a comparison of these two entities between different solutions, the algorithm selects excellent individuals to form the new parent population, and generates a new offspring population through the basic operation (crossover operator, mutation operator, insert operator, and straight operator [13,14,24]) of the genetic algorithm. The Pareto front is constantly optimized by successive generations until the convergence condition is satisfied, as shown in Fig. 4.

All the solutions on the final Pareto front can be regarded as optimal solutions, considering that there are no other solutions with a better simultaneous safety and cost performance. However, unlike other objectives such as environmental impact [50] and travel time [17], driver and passenger safety is a complex but vital problem that cannot be evaluated based solely on the cost–benefit analysis. All these solutions must therefore be further examined by a risk assessment framework to determine whether they fall within the range of acceptable risk.

To determine and limit the risk level, the Dutch Ministry of Housing, Spatial Planning, and Environment (VROM) proposed a framework for risk evaluation [51,52], which was further improved by later scholars [53–56]. *Acceptable risk* has two implications, according to the perspective of the decision-makers. The first implication is the personally acceptable level of risk, which is defined as "the frequency at which an individual may be expected to sustain a given level of harm from the realization of a specific hazard" [55]; this can be calculated in the following way [54]:

$$p_f = \frac{\hat{a}_i}{10^4 P_{d|f}} \tag{17}$$

where p_f denotes the personally acceptable level of risk and $P_{d|f}$ is the death rate in a traffic accident, which is typically about 0.4%–10% [57–59]; here, we assume it to be a conservative 10%. $\hat{a}_i$ is a policy factor that reflects the public attitude toward the acceptable risk level for activity i, and varies from 0.01 (in the case of the most prudent attitude and the strongest tendency for risk-aversion strategies, such as in liquefied petroleum gas (LPG)-station construction) to 100 (in a case with complete freedom of choice, such as in mountaineering); $\hat{a}_i$ should be 1.0 in the general case of road safety [55]. From a personal perspective, the probability of a severe traffic accident involving fatalities should meet the following requirement:

$$P_{fi} < p_f \tag{18}$$

Table 3
Description of significant explanatory variables [5].

Categories	Variable X_i	Description
Traffic situation	AADT ($\mathrm{veh \cdot d^{-1}}$)	Annual average daily traffic
Driver error	*Li*	Lighting design in tunnel
	L_{tu} (km)	Tunnel length
	L_{tan} (km)	Tangent length
	Ve	Ventilation design in tunnel
Vehicle-operating problems	ΔV_{d} ($\mathrm{km \cdot h^{-1}}$)	Operating speed consistency on single element, $\Delta V_{\mathrm{d},i} = V_{\mathrm{o},i} - V_{\mathrm{d}}$, where $\Delta V_{\mathrm{o},i}$ is the operating speed and V_{d} is the design speed in the observed segment i
	ΔV_{o} ($\mathrm{km \cdot h^{-1}}$)	Operating speed consistency on successive elements, $\Delta V_{\mathrm{o},i} = V_{\mathrm{o},i} - V_{\mathrm{o},i-1}$
	Δf_{r}	Vehicle stability, the difference between the skid resistance assumed (f_{ra}) and demanded (f_{rd}); $\Delta f_{\mathrm{r}} = (f_{\mathrm{ra}} - f_{\mathrm{rd}})C$, where $C = 1$ if pavement surface material is concrete, $C = 0.5$ if pavement surface material is asphalt
	L_{s} (km)	Large steep slope (slope grade > 2%), $L_{\mathrm{s}} = LG$, interaction between slope grade G (%), and length of slope segment L (km)
	S_{d} (m)	Stopping sight distance, $S_{\mathrm{d}} = 0.278 V_{\mathrm{o},i} t_{\mathrm{r}} + 0.039 V_{\mathrm{o},i}^2 / a$, where t_{r} is the brake reaction time that the AASHTO assumes to be 2.5 s in open roads and 3.0 s in tunnels, and a is the deceleration rate ($3.4\ \mathrm{m \cdot s^{-2}}$)
Weak fault tolerance	*Sh*	Road shoulder width
	Em	Distance between emergency stopping strips

$V_{\mathrm{o},i} = 135.490 - 7.483/R_i - 1.29G - 14.427Tu - 4.083Br$, where $Tu(Br)$ is the binary variable, which is equal to 1 if the segment is on a tunnel (bridge): $f_{\mathrm{ra}} = 0.22 - 1.79 \times 10^{-3} V_{\mathrm{d}} + 0.56 \times 10^{-5} V_{\mathrm{d}}^2$, $f_{\mathrm{rd}} = V_{\mathrm{o},i}/(127R) - e$, where e is the superelevation.

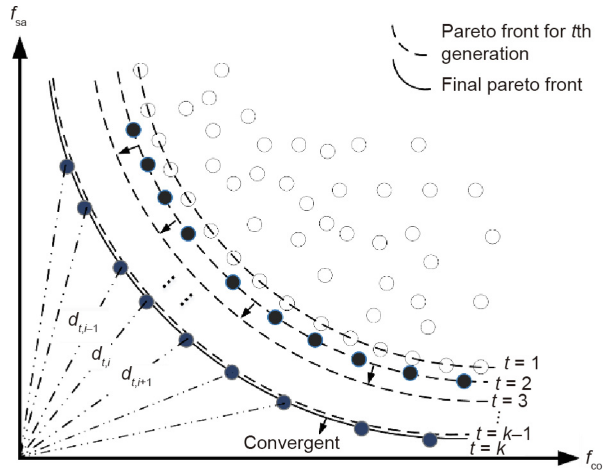


Fig. 4. Final Pareto front generation process.

where P_{fi} is the probability of an individual suffering an accident on the highway, and its calculation is based on the predicted accident frequency f_{sa} (acc·km^{-1}·a^{-1}) for this section of the highway:

$$P_{fi} = \frac{f_{eq}}{AT} = \frac{24 \times f_{sa} \times V_d}{365 \times AADT \times L} \tag{19}$$

where V_d is the design speed, L is the total length of the road, and AT is the annual traffic. Given that drivers (i.e., users) only spend a portion of each day ($L/24V_d$) on this section of the highway, the equivalent annual accident frequency f_{eq} should be increasing the f_{sa} by this portion.

In addition to personally acceptable risk, a socially (nationally) acceptable level of risk is an important consideration that reflects the risk evaluation from a national perspective. To meet the requirements of this aspect, the following conditions should be implemented [55]:

$$E(N_{di}) + R_a(N_{di}) < 7 \times 10^{-6} \hat{a}_i P_N \tag{20}$$

$$E(N_{di}) = N_{Ai} f_{sa} P_{d|f} \tag{21}$$

$$R_a(N_{di}) = 3P_{d|f}\sqrt{N_{Ai} f_{sa}(1 - f_{sa})} \tag{22}$$

where $E(N_{di})$ represents the expected number of fatalities in traffic accidents in the country, $R_a(N_{di})$ is the impact of risk aversion by society toward rare accidents with large fatalities, P_N is the national population size, and N_{Ai} is the total length of highway in the country (which is about 1.31×10^5 km in China).

If both Eqs. (18) and (20) are satisfied for a proposed solution's safety performance, that solution is perceived as falling within the range of acceptable risk. In practice, it is only necessary to compare the predicted risk level of the proposed solution with the lower values of Eqs. (18) and (20). All non-dominated solutions should be examined by the risk assessment program in order to rule out solutions with unacceptable risk in terms of traffic safety (see Fig. 5).

3.2. Marginal efficiency analysis of safety improvement

Given the residual solutions, decision-makers are recommended to adopt one according to their preference. This paper provides three commonly used decision-making perspectives. The first scenario is to choose a solution with the highest traffic safety performance as the optimal solution; this decision-making method is

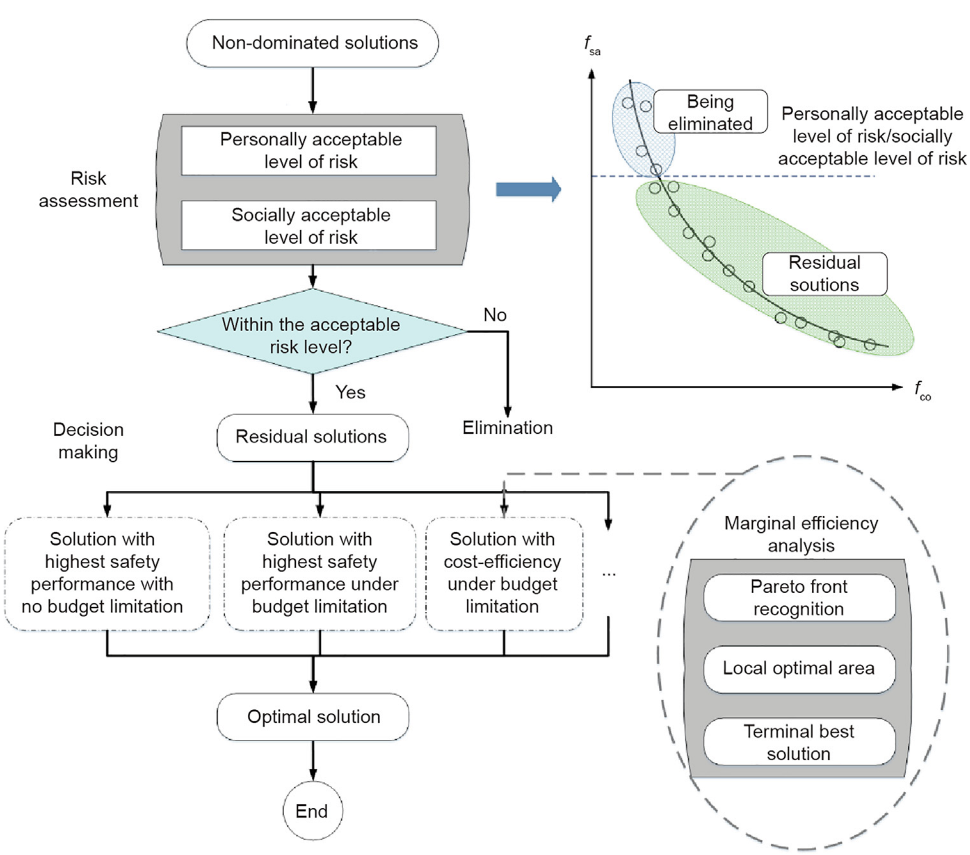


Fig. 5. Risk assessment for decision-making in highway alignments.

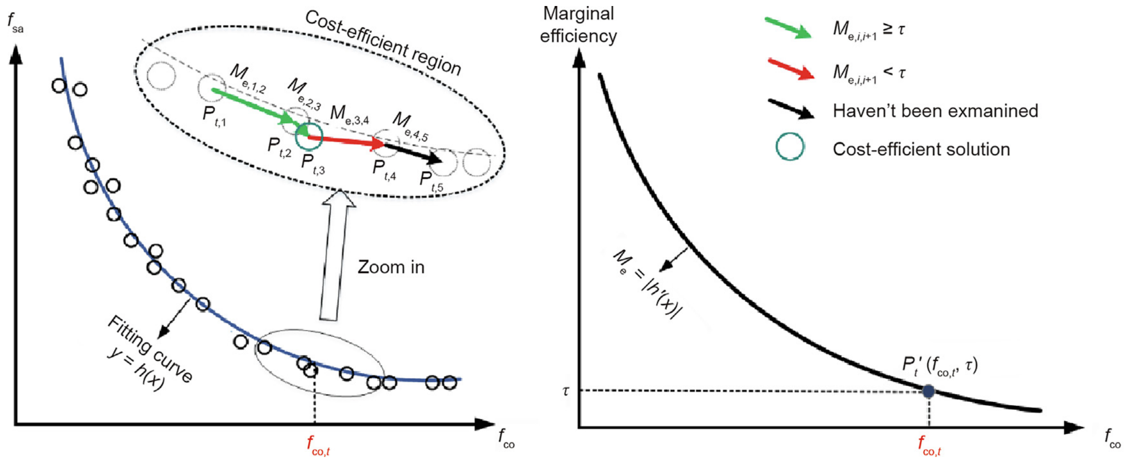


Fig. 6. Cost-efficient solution selection.

based on the assumption of a limitless budget. The second scenario is to choose a solution with the highest traffic safety performance under the budget limitation. The third option is to choose a cost-efficient solution based on a marginal efficiency analysis; this option is detailed in the following paragraphs of this section. It is noted that all of the residual solutions are feasible, and the above three solutions are representative examples.

Given the residual solutions within the budget, an algorithm for selecting a cost-efficient solution based on a marginal efficiency analysis is provided in the following section. *Marginal efficiency* refers to the increase in safety performance that results from extra investment. Considering the law of diminishing marginal utility [60], which is depicted in Fig. 6, a cost-efficient point exists in the Pareto front that satisfies the requirement of minimal marginal efficiency.

Suppose a fitting curve for the Pareto front is represented by $y = h(x)$. The approximate marginal efficiency M_e of each point on the Pareto front can be calculated as shown in Eq. (23). Since the Pareto front can be regarded as a decision for safety improvement at the expense of extra investment, a decision criterion to select the cost-efficient solution from the Pareto set is defined as follows:

$$M_e = \left|\frac{dy}{dx}\right| = |h'(x)| \tag{23}$$

It can be seen from Fig. 6 that the marginal efficiency of the safety improvement is remarkable at the lower cost, but decreases as the cost investment increases, and gradually grows to 0. More specifically, even a substantial increase in investment will only minimally improve safety when the cost performance exceeds a certain threshold ($M_e \geq \tau$). Point P_t', which completes $M_e = \tau$, is seen as a turning point for the cost–benefit balance, and is also regarded as the cost-efficient solution for safety improvement.

Since the fitting curve only reflects the overall shape, rather than the local distribution of the Pareto set, a two-stage filtering algorithm is applied for the marginal efficiency analysis.

Step 1: A curve-fitting method is employed to exhibit the Pareto front through a Matlab tool, and then the fitting curve is transformed into a marginal efficiency curve through derivatives, as in Eq. (23). A predefined threshold τ (which is set at 0.5 in this paper) is used to find the local region in which the best solution lies. The local region is defined as the five points $P_{t,i} (i \in 1, 2, \ldots, 5)$ closest to P_t' among the non-dominated solutions, where $P_t'\left(f_{co,t}, f_{sa,t}'\right)$ is subject to $M_e(f_{co,t}) = \tau$.

Step 2: The five points $P_{t,i}$ are used as a sequence to calculate their marginal efficiency $M_{e,i,i+1} = (f_{sa,i+1} - f_{sa,i}) / (f_{co,i+1} - f_{co,i})$ $(i = 1, 2, 3, 4)$. Every improvement $P_{t,i} \rightarrow P_{t,i+1}$ that satisfies $M_{e,i,i+1} \geq \tau$ is considered worthwhile. Thus, the cost investment is increased step by step until $M_{e,i,i+1} < \tau$. Finally, the last improved point $P_{t,i}$ is chosen as the cost-efficient solution for decision-making.

4. Case study

The proposed method was applied to the highway alignment design across Damala Mountain, one of the most difficult sections of the northern route of the China Sichuan–Tibet Highway. The start and end points of this section are Qamdo (97°10′18″ E, 31°08′34″ N, 3283.000) and Tuoba Township (97°31′33″ E, 31°17′07″ N, 4000.000), respectively. As shown in Fig. 7, an existing country road (G317) connected these two locations through the valley. In order to reduce the travel time and enlarge the traffic capacity, a new highway alignment is being designed for this area. Four planning and design sets have been proposed by the designers, which are displayed in Fig. 7. Corridor A is excluded because it is basically the same as the existing road and does not satisfy the constraint of maximum length ($\leq$ 80 km). The remaining three corridors (B, C, and D) proposed by designers represent the initial population and input into the NSGA-II algorithm.

4.1. Data preparation

Three types of data were required in this study: terrain elevation data, crash data from the study area, and the cost and other model-related parameters.

First, a digital elevation model (DEM) of the study area was developed using Global Mapper to transform the terrain data of

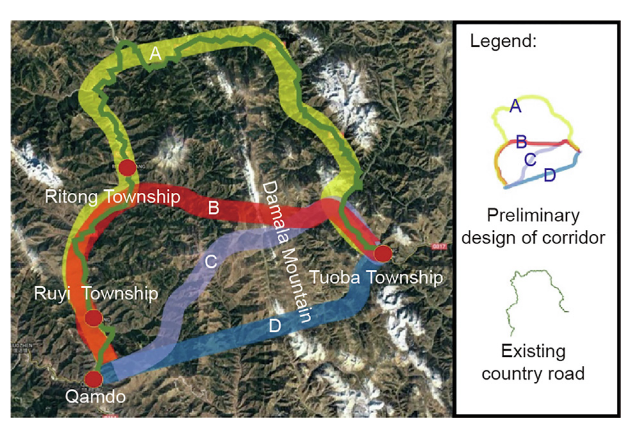


Fig. 7. Background of the study area.

the study area into point-cloud data, which stores the elevation data of every planar point. Next, the point-cloud data were input to the Matlab software in order to reconstruct a DEM (Fig. 8).

Second, crash data (including road and traffic-related factors) of the study area were collected from the local highway administration ministry. A total of 1375 crashes on this highway section were recorded from 2014 to 2017. The regression coefficients were estimated by means of the maximum likelihood method, and the results are shown in Table 4.

Third, the cost and other model-related parameters were collected, and are specified in Table 5.

4.2. Analysis of the results

The combination of DEM in the study area and the initial population constitute the input data, which is the basic required information to start the entire program. As the optimization procedure runs for hundreds of generations, the NSGA-II explores a variety of possibilities for the values of the design variables. Excellent individuals are preserved and selected as the parent alignments to generate new populations through genetic algorithm operators, while poor individuals are eliminated. Fig. 9 shows the searched positions of the intersection points by the NSGA-II. The surrounding space along the local optimal corridors has been carefully exploited in both the horizontal and vertical alignments.

The safety performance and cost performance are gradually improving, which demonstrates the great potential of the NSGA-II for performance optimization. Fig. 10 shows the process of safety performance improvement. Safety-related indicators, such as ΔV_d, ΔV_o, Δf_r, L_s, and S_d, have been gradually improved with the iteration process, resulting in a remarkable decrease in expected traffic accidents. The global variables, such as *Li*, *Ve*, *Sh*, and *Em*, converge earlier than other local variables, which proves that these safety-improvement measures are necessary. Fig. 11 presents the improvement in the expected annual costs of the generated alignments, as well as the improvement details of their components,

Table 4
Estimation results for regression coefficients.

Categories	Variable X_i	Coefficients
Traffic situation	AADT	0.935
Driver error	*Li*	18.745
	L_{tu}	1.136
	L_{tan}	0.697
	Ve	4.124
Vehicle-operating problems	ΔV_d	0.018
	ΔV_o	0.037
	Δf_r	4.692
	L_s	9.473
	S_d	0.091
Weak fault tolerance	*Sh*	16.798
	Em	6.364

Table 5
Cost and other model parameters.

Cost type	Cost items	Values
Construction cost	Ground cut ($RMB{\cdot}m^{-3}$)	$K_{GC} = 54$
	Ground-fill ($RMB{\cdot}m^{-3}$)	$K_{GF} = 34$
	Base price of pavement ($RMB{\cdot}m^{-1}$)	$K_{PA,0} = 8750$
	Base price of bridge ($RMB{\cdot}m^{-1}$)	$K_{BC,0}(H \le 20\ m) = 1.6 \times 10^5$ $K_{BC,0}(20\ m < H \le 40\ m) = 2.0 \times 10^5$ $K_{BC,0}(40\ m < H \le 60\ m) = 2.5 \times 10^5$ $K_{BC,0}(60\ m < H \le 80\ m) = 3.0 \times 10^5$
	Base price of tunnel ($RMB{\cdot}m^{-1}$)	$K_{TC,0} = 3.0 \times 10^5$
	Highway appurtenance ($RMB{\cdot}m^{-1}$)	$K_{FC} = 7300$
	Additional cost compared to $K_{X,0}$ ($RMB{\cdot}m^{-1}$)	$K'_{TC,Li=1} = 900$ $K'_{TC,Li=2} = 1.52 \times 10^6$(RMB per tunnel) $K'_{TC,Ve=1} = 400$ $K'_{X,Pa=1} = 2430$ $K'_{X,Sh=1} = 2250$ $K'_{X,Sh=2} = 1070$ $K'_{X,Em=1} = 457$
Maintenance cost	Maintenance cost ($RMB{\cdot}m^{-1}{\cdot}a^{-1}$)	$K_M = 500$
	Base cost of tunnel operation ($RMB{\cdot}m^{-1}{\cdot}a^{-1}$)	$K_{O,0} = 2000$
	Additional cost compared to $K_{O,0}$ ($RMB{\cdot}m^{-1}{\cdot}a^{-1}$)	$K'_{O,Li=1} = 200$ $K'_{O,Li=2} = 3.02 \times 10^5$ $K'_{O,Ve=1} = 108$

H: bridge pier's height.

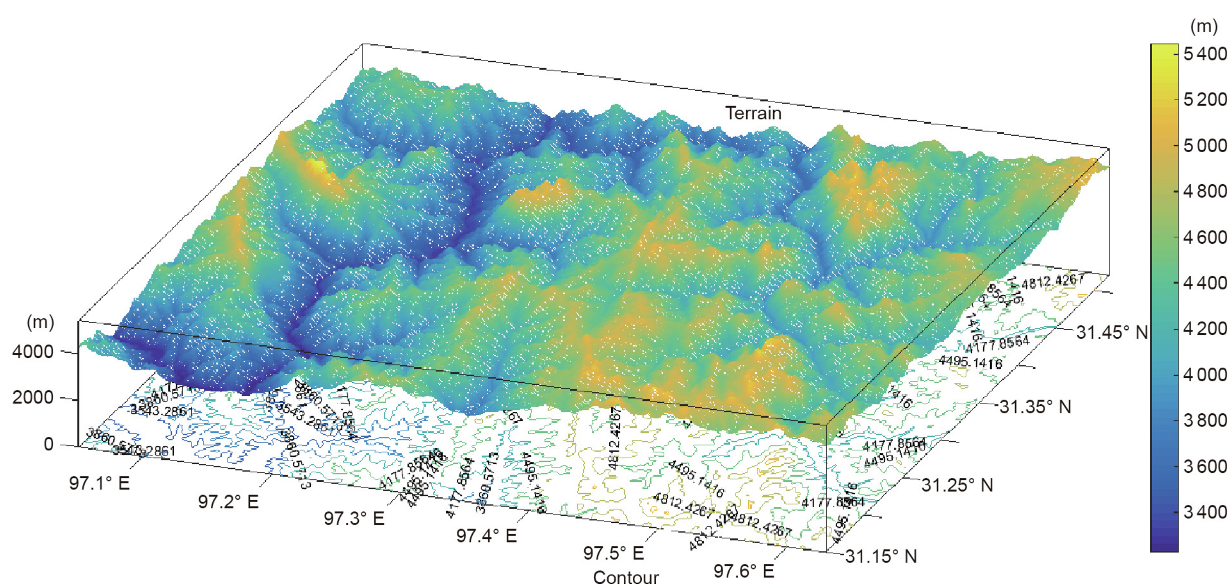


Fig. 8. Digital elevation model of the study area.

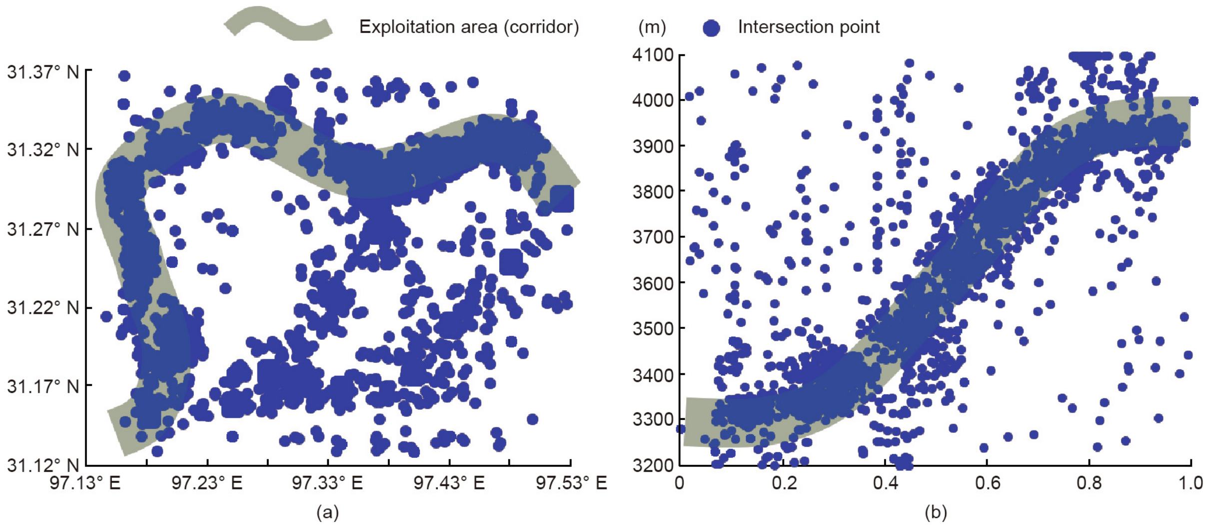


Fig. 9. Searching space for highway alignment optimization.

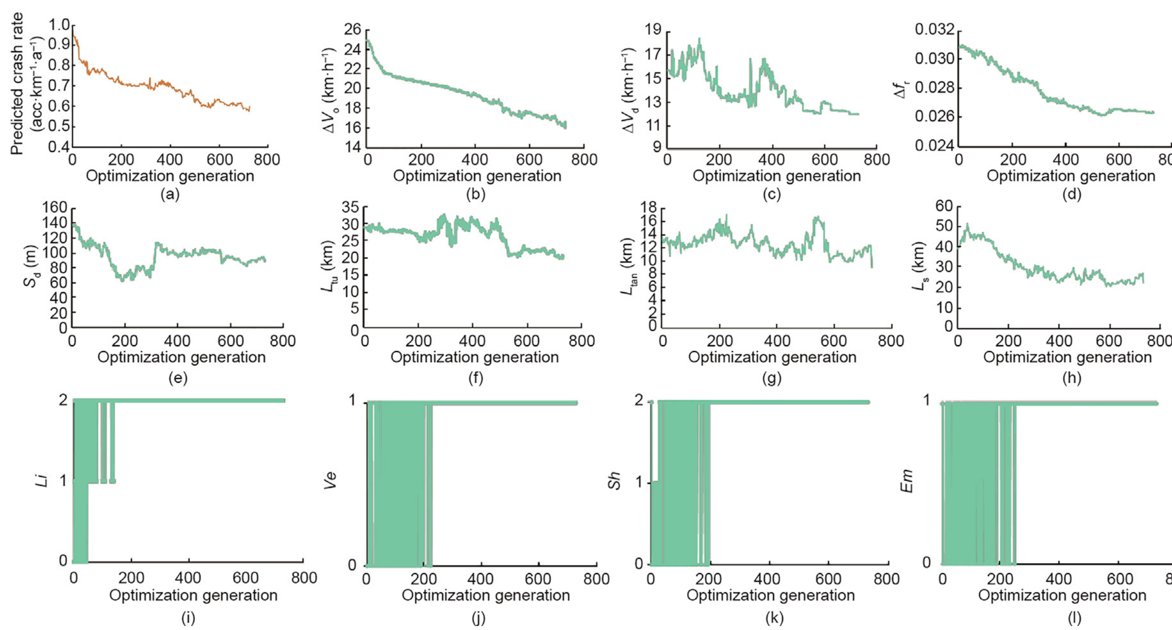


Fig. 10. The transition of the safety performance during the search. (a) Safety performance; Explanatory variables including (b) ΔV_o, (c) ΔV_d, (d) Δf_r, (e) S_d, (f) L_{tu}, (g) L_{tan}, (h) L_s, (i) *Li*, (j) *Ve*, (k) *Sh*, (l) *Em*.

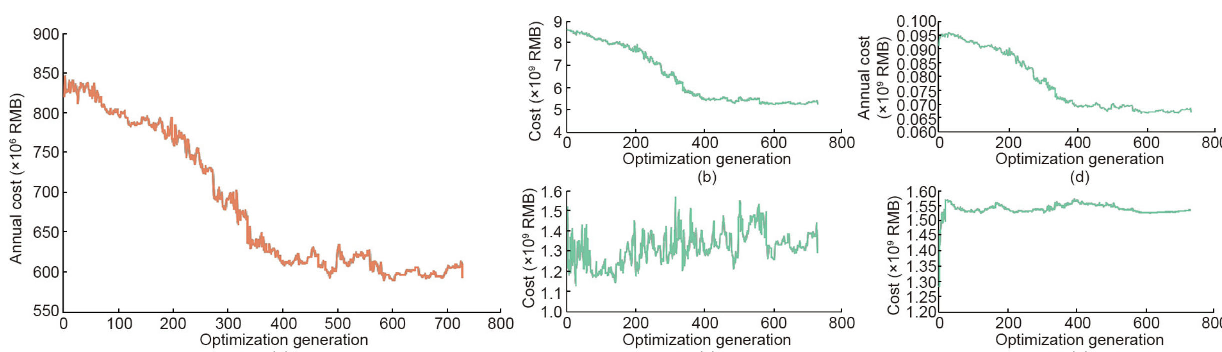


Fig. 11. The transition of the cost performance during the search. (a) Cost performance; (b) construction cost of tunnel; (c) construction cost of bridge; (d) maintenance cost; (e) construction cost of road.

including construction costs for various highway structures and maintenance costs. Note that Figs. 10 and 11 only show the average value of part of the indicators from a certain generation.

In the search, it was gradually discovered that corridor B has the greatest potential for optimization of the three preliminary corridors (B, C, and D) since it has the minimum tunnel length, which results in a slump in cost. Fig. 12 shows the performance results of the optimization generations of the search program. The traffic safety and construction cost are optimized simultaneously in the iteration. The program terminated at gen#729, when the convergent condition was reached. The personally acceptable safety performance $f_{\mathrm{sa}}^{\mathrm{p}} = 52.731\ \mathrm{acc} \cdot \mathrm{km}^{-1} \cdot a^{-1}$ and the nationally acceptable safety performance $f_{\mathrm{sa}}^{\mathrm{n}} = 0.744\ \mathrm{acc} \cdot \mathrm{km}^{-1} \cdot a^{-1}$ were calculated according to Eqs. (19) and (20). It was found that all the non-dominated solutions at gen#729 fell within the personally and nationally acceptable risk levels, so no solutions were eliminated because they exceeded the safety threshold.

There are three optional optimal solutions, as shown in Fig. 12: Optimal solution 1 has the best safety performance without considering a budget limitation, optimal solution 2 has the best safety performance considering a budget limitation of 700 million RMB, and optimal solution 3 is the cost-efficient solution within the budget limitation. Information on these three solutions is provided in Fig. 13; this information includes feasible and commonly used recommendations for decision-makers. Decision-makers are encouraged to adopt one of these solutions according to the practical situation and their preference.

5. Discussion and conclusion

The optimization results show that the proposed model is capable of generating high-quality solutions, and can achieve low costs and low accident rates. However, it was found that the nationally acceptable safety performance was far lower than the

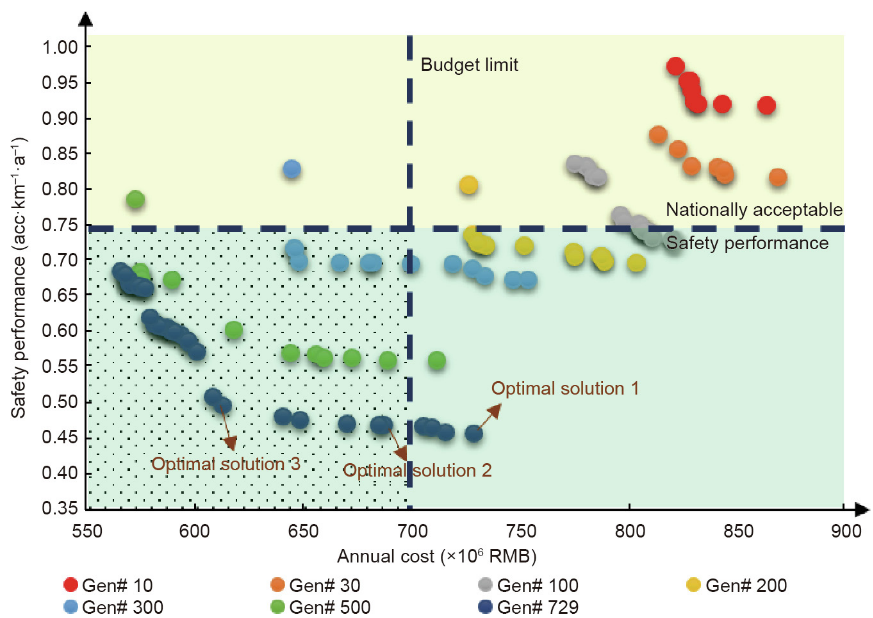
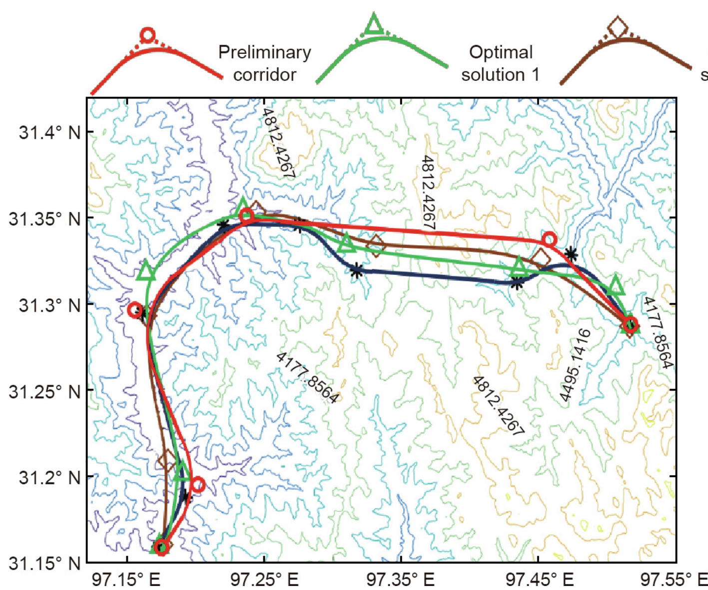


Fig. 12. Performance results of optimization generations.



Alignment	Preliminary corridor	Optimal solution 1	Optimal solution 2	Optimal solution 3
Length (km)	57.04	61.63	62.56	59.07
Average gradient (%)	2.33	1.40	1.46	1.49
Average curve radius (m)	972.4	823.2	986.2	1076.5
Ratio of bridges and tunnels (%)	62.59	48.61	47.76	45.52
The length of bridge (km)	6.15	9.55	9.44	9.20
The length of tunnel (km)	29.55	20.41	20.44	17.69
The length of road (km)	21.34	31.67	32.68	32.18
Construction cost (×10⁶ RMB)	11.55	10.04	9.39	8.37
Maintenance cost (×10⁶ RMB)	93.5	75.7	76.2	68.4
Annual cost (×10⁶ RMB)	844.2	728.3	686.9	612.6
Safety performance	0.991	0.458	0.469	0.498

Fig. 13. Computer-generated optimal solution with respect to cost and safety.

personally acceptable safety performance. This is because, for individuals, the possibility of being killed on this highway is relatively low—even though many traffic accidents happen, most are property damage only (PDO) or crashes that cause injuries [31]; thus, the estimated number of fatalities is not particularly high when compared with aircraft accidents [55]. However, for society (i.e., the nation), the highway death rate (fatalities/population), which does not take the total highway length of a country into consideration, is an important index for evaluating traffic safety [61,62]. Considering that the total length of all Chinese highways is very large, the permissible accident rate per kilometer must be seriously limited in order to lower the total death rate on all highways at the national level. It should be noted that the nationally acceptable risk levels are different for each country, and the personally acceptable risk levels of different countries (i.e., different death rates in cases of traffic accidents) are also different. Therefore, both acceptable risk levels must be computed, since it is not certain which one is lower (dominating).

Throughout the experiment, the optimization process and the scope of the searching space were displayed. In the NSGA-II, 143 314 offspring were generated during the search. The mean annual cost value of the final generation was 610.08 million RMB, while the mean cost value of the first generation was 821.17 million RMB, which is 1.35 times higher. Similarly, the mean safety performance value of the final generation was 5.84, while the mean value of the first generation was 9.74, which is 1.67 times higher. Fig. 10 shows that the safety optimization of the planning and design was mainly achieved by reducing the length of tunnels and steep slopes and improving the design consistency. The quick convergence of the global variables may indicate that the improvements in the lighting, ventilation, road width, and emergency stopping strips design are valuable, since they significantly improve operation safety at a low cost. Fig. 11 shows that cost optimization was mainly achieved by replacing the tunnel structures with road structures.

This paper presents a novel framework that strikes a balance between cost and safety for mountainous highway alignment design. The contribution of this paper to the highway alignment problem is as follows: ① An FTA was combined with regression analysis methods to model the relations between traffic accidents and highway design variables. An NFTA diagram was presented in order to identify the risk factors affecting both operation safety and life-cycle cost among the design variables. Next, a generalized linear-regression model was introduced in order to evaluate the safety performance of a highway alignment. ② The highway planning and design variables were arrayed in a new form of chromosome genes, which formed the basis of the cost and safety analysis. ③ The relations between the life-cycle cost of the highway planning and design and the states of the risk factors were developed in the cost-assessment functions. The impact of risk factors on the life-cycle cost was modeled by Eqs. (9)–(12). ④ The theory of acceptable risk was introduced to discuss individual and social attitudes toward highway safety in the proposed alignments. ⑤ A marginal efficiency-based computing method was proposed in order to solve for the cost-efficient solution among the candidate optimal solutions.

Despite all the effort described above, there is still room for future improvement. Since the SPF model was trained using crash data collected from the same region as the engineering project, it is suggested that other scholars not apply this model directly to other regional projects. The neglect of spatial heterogeneity might lead to biased parameters and model misspecification. Spatially varying coefficients models can be investigated in order to account for unstructured and spatially structured heterogeneity in future research.

In conclusion, this paper identifies the relations between life-cycle cost and traffic safety in mountainous highway construction projects, and strikes an optimal balance between them within acceptable risk levels. The proposed method can be applied to a reasonable investment analysis and multi-objective optimization in other high-safety-risk engineering projects.

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Compliance with ethics guidelines

Chengqian Li, Lieyun Ding, and Botao Zhong declare that they have no conflict of interest or financial conflicts to disclose.

References

[1] Chen F, Chen S. Differences in injury severity of accidents on mountainous highways and non-mountainous highways. Procedia Soc Behav Sci 2013;96:1868–79.
[2] Zhao H, Yin Z, Xiang H, Liao Z, Wang Z. Preliminary study on alterations of altitude road traffic in China from 2006 to 2013. PLoS ONE 2017;12(2): e0171090.
[3] Ma S, Shao C, Zhai R, Liu D. Traffic safety evaluation of the provincial regions in china based on principal component analysis. In: Proceedings of the 2009 Second International Conference on Intelligent Computation Technology and Automation; Oct 10–11; Changsha, China. New York: IEEE; 2009. p. 864–7.
[4] Mainwaring G, Olsen TO. Long undersea tunnels: recognizing and overcoming the logistics of operation and construction. Engineering 2018;4(2):249–53.
[5] Montella A, Imbriani LL. Safety performance functions incorporating design consistency variables. Accid Anal Prev 2015;74:133–44.
[6] Ma Z, Zhang H, Chien SIJ, Wang J, Dong C. Predicting expressway crash frequency using a random effect negative binomial model: a case study in China. Accid Anal Prev 2017;98:214–22.
[7] Zhang Y, Chini A. Performance of seven highway construction contracting methods analyzed by project size. Front Eng Manage 2018;5(2):1–11.
[8] Ismail K, Sayed T. Risk-optimal highway design: methodology and case studies. Saf Sci 2012;50(7):1513–21.
[9] Rosey F, Auberlet JM. Trajectory variability: road geometry difficulty indicator. Saf Sci 2012;50(9):1818–28.
[10] Kang MW, Jha MK, Schonfeld P. Applicability of highway alignment optimization models. Transp Res Part C Emerg Technol 2012;21(1):257–86.
[11] Hare W, Hossain S, Lucet Y, Rahman F. Models and strategies for efficiently determining an optimal vertical alignment of roads. Comput Oper Res 2014;44:161–73.
[12] Cheng JF, Lee Y. Model for three-dimensional highway alignment. J Transp Eng 2006;132(12):913–20.
[13] Jong J-C, Jha MK, Schonfeld P. Preliminary highway design with genetic algorithms and geographic information systems. Computer-Aided Civil Infrastruct Eng 2000;15(4):261–71.
[14] Li W, Pu H, Schonfeld P, Yang J, Zhang H, Wang L, et al. Mountain railway alignment optimization with bidirectional distance transform and genetic algorithm. Comput Aided Civ Infrastruct Eng 2017;32(8):691–709.
[15] Jha MK, Schonfeld P. A highway alignment optimization model using geographic information systems. Transp Res Part A Policy Pract 2004;38 (6):455–81.
[16] Kang MW, Schonfeld P, Yang N. Prescreening and repairing in a genetic algorithm for highway alignment optimization. Comput Aided Civ Infrastruct Eng 2009;24(2):109–19.
[17] Maji A, Jha MK. Multi-objective highway alignment optimization using a genetic algorithm. J Adv Transp 2009;43(4):481–504.
[18] Kang MW. An alignment optimization model for a simple highway network [dissertation]. College Park: University of Maryland; 2008.
[19] Ding L, Xu J. A review of metro construction in China: organization, market, cost, safety and schedule. Front Eng Manage 2017;4(1):4–19.
[20] Hammad A, Itoh Y, Nishido T. Bridge planning using GIS and expert system approach. J Comput Civ Eng 1993;7(3):278–95.
[21] Kim E, Jha MK, Son B. Improving the computational efficiency of highway alignment optimization models through a stepwise genetic algorithms approach. Transp Res Part B: Methodol 2005;39(4):339–60.
[22] Pushak Y, Hare W, Lucet Y. Multiple-path selection for new highway alignments using discrete algorithms. Eur J Oper Res 2016;248(2):415–27.

[23] Shafahi Y, Bagherian M. A customized particle swarm method to solve highway alignment optimization problem. Comput Aided Civ Infrastruct Eng 2013;28(1):52–67.
[24] Jong JC, Schonfeld P. An evolutionary model for simultaneously optimizing three-dimensional highway alignments. Transp Res Part B: Methodol 2003;37(2):107–28.
[25] Saha P, Ksaibati K. An optimization model for improving highway safety. J Traffic Transp Eng 2016;3(6):549–58.
[26] Rodriguez-Roman D. A surrogate-assisted genetic algorithm for the selection and design of highway safety and travel time improvement projects. Saf Sci 2018;103:305–15.
[27] Kang MW, Shariat S, Jha MK. New highway geometric design methods for minimizing vehicular fuel consumption and improving safety. Transp Res Part C Emerg Technol 2013;31:99–111.
[28] Casal G, Santamarina D, Vázquez-Méndez ME. Optimization of horizontal alignment geometry in road design and reconstruction. Transp Res Part C Emerg Technol 2017;74:261–74.
[29] Ahmed M, Huang H, Abdel-Aty M, Guevara B. Exploring a Bayesian hierarchical approach for developing safety performance functions for a mountainous freeway. Accid Anal Prev 2011;43(4):1581–9.
[30] Zheng Z, Du Z, Yan Q, Xiang Q, Chen G. The impact of rhythm-based visual reference system in long highway tunnels. Saf Sci 2017;95:75–82.
[31] Huang H, Peng Y, Wang J, Luo Q, Li X. Interactive risk analysis on crash injury severity at a mountainous freeway with tunnel groups in China. Accid Anal Prev 2018;111:56–62.
[32] Zhao Y, Li P. A statistical analysis of China's traffic tunnel development data. Engineering 2018;4(1):3–5.
[33] Kabir S. An overview of fault tree analysis and its application in model based dependability analysis. Expert Syst Appl 2017;77:114–35.
[34] Bobbio A, Portinale L, Minichino M, Ciancamerla E. Improving the analysis of dependable systems by mapping fault trees into Bayesian networks. Reliab Eng Syst Saf 2001;71(3):249–60.
[35] Ng JCW, Sayed T. Effect of geometric design consistency on road safety. Can J Civ Eng 2004;31(2):218–27.
[36] Cheliyan AS, Bhattacharyya SK. Fuzzy fault tree analysis of oil and gas leakage in subsea production systems. J Ocean Eng Sci 2018;3(1):38–48.
[37] Hirpa D, Hare W, Lucet Y, Pushak Y, Tesfamariam S. A bi-objective optimization framework for three-dimensional road alignment design. Transp Res Part C Emerg Technol 2016;65:61–78.
[38] Cafiso S, La Cava G. Driving performance, alignment consistency, and road safety: real-world experiment. Transp Res Rec 2009;2102(1):1–8.
[39] Da Costa JO, Jacques MAP, Soares FEC, Freitas EF. Integration of geometric consistency contributory factors in three-leg junctions collision prediction models of Portuguese two-lane national highways. Accid Anal Prev 2016;86:59–67.
[40] AASHTO. Highway Safety Manual. Washington: AASHTO; 2010.
[41] Brimley B, Saito M, Schultz G. Calibration of Highway Safety Manual safety performance function: development of new models for rural two-lane two-way highways. Transp Res Rec 2012;2279(1):82–9.
[42] Venkataraman NS, Ulfarsson GF, Shankar VN. Extending the Highway Safety Manual (HSM) framework for traffic safety performance evaluation. Saf Sci 2014;64:146–54.
[43] U. S. Department of Transportation Federal Highway Administration. Interactive Highway Safety Design Model (IHSDM). Overview. Version14.0.0 [software]. 2018 Sep [cited 2018 Oct 24]. Available from: https://highways.dot.gov/safety/interactive-highway-safety-design-model/interactive-highway-safety-design-model-ihsdm.
[44] AASHTO. Safety analyst overview. [Software]. 2018 [cited 2018 Oct 24]. Available from: http://www.safetyanalyst.org.
[45] U. S. Department of Transportation Federal Highway Administration. Crash modification factors clearinghouse. 2018 [cited 2018 Nov 20]. Available from: http://www.cmfclearinghouse.org.
[46] Li L, Gayah VV, Donnell ET. Development of regionalized SPFs for two-lane rural roads in Pennsylvania. Accid Anal Prev 2017;108:343–53.
[47] Donoghoe MW, Marschner IC. Stable computational methods for additive binomial models with application to adjusted risk differences. Comput Stat Data Anal 2014;80:184–96.
[48] Gomes MJTL, Cunto F, da Silva AR. Geographically weighted negative binomial regression applied to zonal level safety performance models. Accid Anal Prev 2017;106:254–61.
[49] Deb K, Pratap A, Agarwal S, Meyarivan T. A fast and elitist multiobjective genetic algorithm: NSGA-II. IEEE Trans Evol Comput 2002;6(2):182–97.
[50] Yang N, Kang MW, Schonfeld P, Jha MK. Multi-objective highway alignment optimization incorporating preference information. Transp Res Part C Emerg Technol 2014;40:36–48.
[51] Ale BJM. Tolerable or acceptable: a comparison of risk regulation in the United kingdom and in the Netherlands. Risk Anal 2005;25(2):231–41.
[52] Bottelberghs PH. Risk analysis and safety policy developments in the Netherlands. J Hazard Mater 2000;71(1–3):59–84.
[53] Jonkman SN, Van Gelder PHAJM, Vrijling JK. An overview of quantitative risk measures for loss of life and economic damage. J Hazard Mater 2003;99(1):1–30.
[54] Vrijling JK, Van Hengel W, Houben RJ. A framework for risk evaluation. J Hazard Mater 1995;43(3):245–61.
[55] Vrijling JK, Van Hengel W, Houben RJ. Acceptable risk as a basis for design. Reliab Eng Syst Saf 1998;59(1):141–50.
[56] Vrijling JK. Probabilistic design of water defense systems in The Netherlands. Reliab Eng Syst Saf 2001;74(3):337–44.
[57] Dharmaratne SD, Jayatilleke AU, Jayatilleke AC. Road traffic crashes, injury and fatality trends in Sri Lanka: 1938–2013. Bull World Health Organ 2015;93(9):640–7.
[58] St Bernard G, Matthews W. A contemporary analysis of road traffic crashes, fatalities and injuries in Trinidad and Tobago. Inj Control Saf Promot 2003;10(1–2):21–7.
[59] Wang SY, Chi GB, Jing CX, Dong XM, Wu CP, Li LP. Trends in road traffic crashes and associated injury and fatality in the People's Republic of China, 1951–1999. Inj Control Saf Promot 2003;10(1–2):83–7.
[60] Li J, Pollard S, Kendall G, Soane E, Davies G. Optimising risk reduction: an expected utility approach for marginal risk reduction during regulatory decision making. Reliab Eng Syst Saf 2009;94(11):1729–34.
[61] Peden M, Scurfield R, Sleet D, Mohan D, Hyder AA, Jarwan E, et al. World report on road traffic injury prevention. Geneva: World Health Organization; 2004.
[62] Kopits E, Cropper M. Traffic fatalities and economic growth. Accid Anal Prev 2005;37(1):169–78.