교차로와 구조물을 고려한 도로선형 최적화 모형 개발

Modeling Intersections and Other Structures for Highway Alignment Optimization

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Abstract

Previous alignment optimization models have not adequately considered intersections and other structures such as bridges, tunnels, grade separations and interchanges which can very strongly affect alignment decisions. This paper develops comprehensive cost functions for intersections and other structures and incorporates them in recently developed highway alignment optimization models connected with genetic algorithms and geographical information systems. The result is a fast and computerized process for extracting, analyzing spatial data, evaluating candidate alignments and optimizing them. A method for locally optimizing intersections is also developed. It improves search flexibility by saving good alignments whose unacceptable crossing angles with existing roads can be fixed. Through case studies, the developed model is found to produce feasible and efficient solutions.

1. INTRODUCTION

Optimization of highway alignments aims to select the best alignment among many alternatives, based on specified objective functions and while satisfying various design constraints. Since a highway alignment is made subject to a set of design constraints and operational requirements, even slight changes in a particular part of an alignment may eventually influence its whole configuration, thus considerably changing the total costs. In addition to these, what if we allow an alignment to have intersections, tunnels, bridges and other structures? And what if we allow the existing roads to be re-optimized for better crossings with the new alignment? We can easily imagine how much these considerations affect the final alignment configuration.

In earlier developments of highway alignment optimization models (during the 60s and 70s), researchers mainly focused on having an algorithm to work within simple study areas. For instance, the new alignments usually did not have existing roads to cross or creeks and rivers to be bridged, or very irregular (mountainous) terrain where tunnels may be more economical than cuts.

Recently in the USA, a new highway is typically needed to reduce traffic congestion by providing a bypass route or enhance accessibility between two or more transportation demand sources while minimizing environmental impacts and community concerns. This trend means that a new highway is more likely to pass through a complex environment and require many structures. Indeed, the outputs from alignment optimization models able to handle structures could differ considerably from those without such capabilities.

There have been three types of models for optimizing highway alignments as listed in Table 1: (1) horizontal alignment optimization models, (2) vertical alignment optimization models and (3) models for simultaneously optimizing horizontal and vertical alignments.

Table 1 Studies on Highway Alignment Optimization

Target for optimizing	Types of approach	References
Horizontal alignment	Calculus of variations	Wan (1995), Howard et al. (1968), Thomson and Sykes (1988), Shaw and Howard (1981 and 1982)
	Network optimization	OECD (1973), Turner and Miles (1971), Athsanassoulis and Calogero (1973), Parker (1977), Trietsch (1987a and 1987b)
	Dynamic programming	Hogan (1973) and Nicholson et al. (1976)
	Genetic algorithms	Jong (1998)
Vertical alignment	Enumeration	Easa (1988)
	Dynamic programming	Puy Huarte (1973), Murchland (1973), Goh et al. (1988) and Fwa (1989)
	Linear programming	ReVille et al. (1997) and Chapra and Canale (1988)
	Numerical research	Hayman (1970), Goh et al. (1988), Robinson (1973), Fwa (1989) and MINERVA (OECD, 1973)
	Genetic algorithms	Jong (1998)
Horizontal and vertical alignment simultaneously	Dynamic programming	Hogan (1973) and Nicholson et al. (1976)
	Numerical research	Chew et al. (1989)
	Two-Stage Optimization	Parker (1977) and Trietsch (1987a)
	Genetic algorithms	Jong (1998)

However, intersections and other structures have not yet been incorporated into any highway alignment optimization method. Without considering structures, a model for highway alignment optimization would have very limited value and be unsuitable for any preliminary design applications.

The scope of this work covers a relatively broad range of structures most likely encountered when optimizing highway alignments. The representative structures on a highway alignment might be intersections, interchanges, bridges and tunnels. Among them, interchanges, bridges and tunnels are the subjects of their own vast research areas. Moreover, in many cases bridges and tunnels dominate a highway planning and construction process in terms of costs and locations. This study is not intended to deal with those completely dominating cases. Therefore, large interchanges, bridges and tunnels are excluded. However, relatively small interchanges and small-scale overpass and underpass structures are considered. This study mainly focuses on two-lane rural highway alignments. However, the cost functions and alignment optimization models being developed can be extended to other kinds of highways with moderate changes.

2. CHARACTERISTICS OF STRUCTURES ON HIGHWAYS

Many studies have classified highway cost items (Winfrey, 1968; Moavenzadeh et al., 1973; OECD, 1973; Watanatada et al., 1987; Wright, 1996; Jong, 1998; Jha, 2000). No consensus was found about classifying highway alignment costs. When optimizing highway alignments, it is important to include all dominating costs that are also sensitive to alignments, regardless of whatever classification methods were employed. A cost is dominating if it accounts for high percentages in the total costs and is sensitive if it changes substantially with slight alignment changes. It is obvious that costs for intersections and other structures are dominating and sensitive to highway alignments when we consider the characteristics of those.

2.1. At-grade intersection characteristics

Among the factors of intersections, those mainly affecting alignments are vehicle speeds (design speed), vertical alignments at the intersection (differences between an existing road and a new road), angle of the intersection, geometric features (topography of the site and cross sections), design hourly turning movements (additional lane need for turning volumes) and sight distances.

The design speed is one of the most important factors in alignment configurations. Especially in curved sections, the design speed determines the lengths of radii and the associated superelevation. Vertical alignments at intersections very significantly affect earthwork cost estimates. Greater elevation differences between an existing road and a new road imply increasingly higher earthwork costs. The intersection angle is also an important factor. As recommended by AASHTO (2001), too acute or oblique crossings should be avoided.

Intersection geometric features, including cross sections and topography of the site, also affect earthwork, drainage and pavement costs. Design-hour turning movements are the factors determining whether additional lanes are needed for smoothness and safety. They also affect accident frequency. Proper sight distances are important for avoiding potential vehicle conflicts at intersections. Sight distances can be used for finding what obstructions around the intersection should be removed. Hence, sight distance information can be employed to estimate right-of-way costs of the intersection.

2.2. Small bridge characteristics

When bridges are dominating the associated alignments, it is generally true that: "a bridge is the key element in a transportation system for three reasons, (1) it controls the capacity of the system, (2) it is the highest cost per mile of the system and (3) if the bridge fails, the system fails" (Barker and Puckett, 1997). Even for the much smaller bridges considered in this study, the last argument is partially true.

To extract highway bridge characteristics affecting alignments, we need to take earthwork volumes into account. Earthwork costs, especially fill volumes, are directly associated with constructing bridges. There should be an economical break-even point between fills and building bridges depending on various site-specific characteristics.

The other important bridge characteristics affecting highway alignments include radii of bridges, span lengths, number of spans, number of piers and heights of piers. Since this study is not limited to a straight bridge, bridges having horizontal curvatures are considered as parts of alignments. This could cost more for a bridge section. However, the total alignment costs may be smaller. Since bridge costs can be separated into superstructure costs and substructure costs (Xanthankos, 1994; O'Connor, 1971), span lengths, number of spans, number of piers and heights of piers are very crucial for estimating bridge costs.

2.3. Characteristics of grade separation structures (overpass and underpass)

New highway alignments may cross many existing roads, using either underpasses or overpasses, depending on the alignment profile. These grade separation structures have similar characteristics to bridges discussed earlier.

There are two additional factors affecting alignments: vertical and lateral clearances. In principle, the minimum lateral clearance from the edge of the traveled way to the face of the protective barrier should be the normal shoulder width (AASHTO, 1994). The required vertical clearance should also be provided. Although vertical clearances of 4.1 m ~ 4.4 m have been adopted by several U.S. states, additional clearance is desirable to compensate for resurfacings, snow, ice accumulation and an occasional slightly overheight load. The recommended minimum clearance is 4.4 m, and the desirable clearance is 5.0 m (AASHTO, 2000).

2.4. Small tunnel characteristics

Among many highway tunnel elements, the most important factors are ventilation for pollutants and consequent adjustment of the air supply and exhaust, lighting for safety and ensuring maximum appropriate speeds, fire life safety provisions for providing refuge from a raging fire or deadly smoke, elaborate traffic surveillance and control systems coordinated with the other system for protected egress of motorists in the event of a fire and access for fire-fighting personnel, and soil types for earthwork and construction process (King and Kuesel, 1996). These elements are functions of several characteristics of a tunnel. Many tunnel characteristics affect their costs. Among them, those characteristics affecting highway alignments include cross sections, clearances, horizontal alignments and grades.

If possible, the tunnel alignment should be straight. If curves are required, the minimum radius is determined by stopping sight distances and acceptable superelevation in relation to design speed. Where shoulders are narrow, horizontal sight distance may be restricted by the proximity of the tunnel sidewall. Usually, passing distances do not apply.

Upgrades in tunnels carrying heavy traffic are preferably limited to 3.5% to reduce ventilation requirements. For long two-lane tunnels with two-way traffic, a maximum grade of 3% is desirable to maintain reasonable truck speed. For downgrade traffic, 4% or more is acceptable. For lighter traffic volumes, grades up to 5% or even 6% have been used for economy's sake (King and Kuesel, 1996).

2.5. Characteristics of interchanges

Interchanges provide exclusive movement of traffic among two or more roadways and are systems of interconnected roadways using grade separation. General design considerations for interchanges are type determination, including the number of structures involved, horizontal and vertical alignments, cross sections and sight distances.

An interchange could be either a system interchange or a service interchange, depending on its role. AASHTO (2000) states that the final configuration of an interchange may be determined by the need for route continuity, uniformity of exit patterns, single exit in advance of the separation structure, elimination of weaving on the main facility, signing potential, and availability for right-of-way.

The general controls for horizontal and vertical alignment and their combination should be adhered to closely. In particular, any relatively sharp horizontal and vertical curves should be avoided (AASHTO, 2000). Generally, design considerations for interchanges address the same requirements as basic highway segments, even if there are some exceptional cases. Therefore, except for some extreme cases such as ramps, all components of interchanges are treated as one part of basic highway segments when developing cost functions.

3. COST FUNCTIONS OF STRUCTURES

3.1. Methodology for Intersection Construction Cost Modeling

The intersection cost modeling method presented here is extracted from Kim et al. (2001, under reviewing), where more detailed approaches can be found. Intersection costs include right-of-way, pavement, earthwork, accident, vehicle operating and user delay costs. Modeling process begins with identifying the crossing point between a new alignment and an existing road. The boundaries separating alignment segments with intersections are then obtained using design standards. Boundary setting insures that costs for approach segments and intersections are not double-counted.

In estimating earthwork costs for new intersections, the basic idea is to obtain the coordinates of the points **A**, **B**, **D**, **E**, **F**, **J**, and $\mathbf{N}_\mathrm{m}$ in Figure.1 where the quadrant of a typical fill intersection is sliced into several pieces. The coordinates of these points are given as:

$$\mathbf{A} = \mathbf{O} + (R - f)\frac{\mathbf{I} - \mathbf{O}}{\left\|\mathbf{I} - \mathbf{O}\right\|} \tag{1}$$

$$\mathbf{B} = \mathbf{O} + R\frac{\mathbf{I} - \mathbf{O}}{\left\|\mathbf{I} - \mathbf{O}\right\|} \tag{2}$$

$$\mathbf{D} = \mathbf{O} + (R - f)\frac{\mathbf{N}_\mathrm{m} - \mathbf{O}}{\left\|\mathbf{N}_\mathrm{m} - \mathbf{O}\right\|} \tag{3}$$

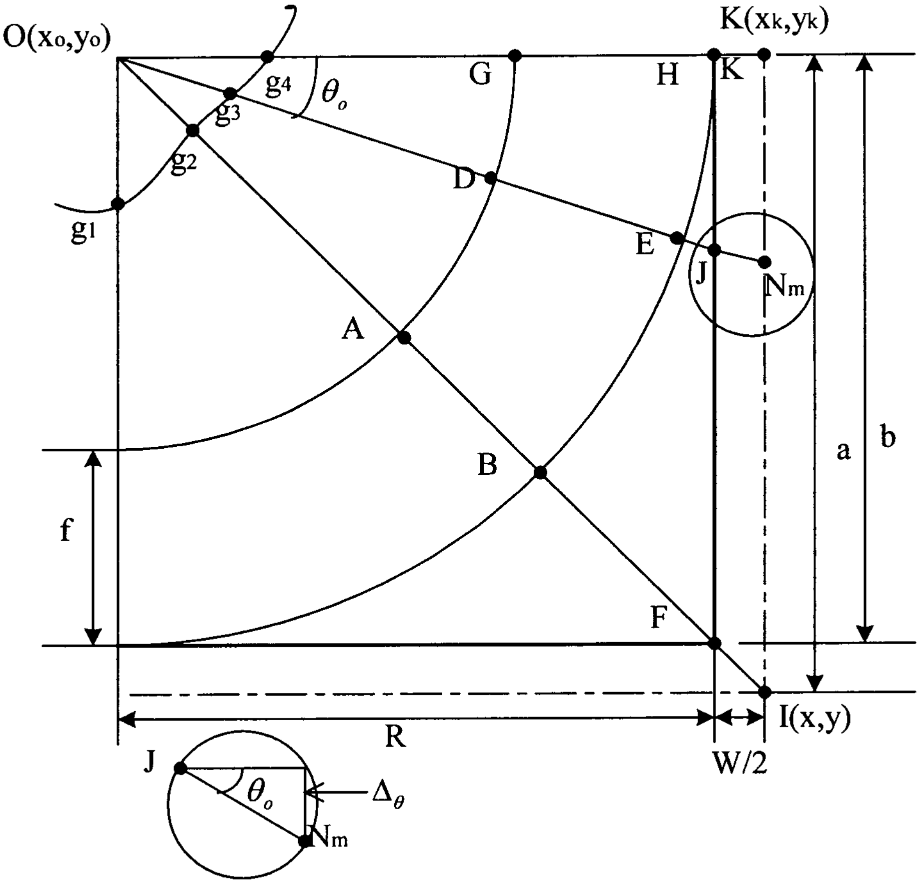


Fig. 1. Important points for determining intersection coordinates (a quadrant of a typical fill intersection)

$$\mathbf{E} = \mathbf{O} + R\frac{\mathbf{N_m} - \mathbf{O}}{\left\|\mathbf{N_m} - \mathbf{O}\right\|} \tag{4}$$

$$\mathbf{F} = \mathbf{O} + \left[\left\|\mathbf{I} - \mathbf{O}\right\| - \frac{W}{\sqrt{2}}\right]\frac{(\mathbf{I} - \mathbf{O})}{\left\|\mathbf{I} - \mathbf{O}\right\|} \tag{5}$$

$$\mathbf{J} = \mathbf{O} + \left[\|\mathbf{N}_{\mathrm{m}} - \mathbf{O}\| - \sqrt{\frac{W^2}{4} + \left(\frac{W}{2} \tan\theta_o \right)^2} \right] \frac{(\mathbf{N}_{\mathrm{m}} - \mathbf{O})}{\|\mathbf{N}_{\mathrm{m}} - \mathbf{O}\|} \qquad (6)$$

$$\mathbf{N}_{\mathrm{m}} = \mathbf{I} + m \frac{\mathbf{K} - \mathbf{I}}{\|\mathbf{K} - \mathbf{I}\|}, \text{ where } 0 \le m \le b \qquad (7)$$

To find the earthwork volumes of each cell in Figure 1, two elevations are needed: (1) base elevation and (2) ground elevation. This study simply averages the associated points' elevations. When there is a total of T parcels in an intersection, the total earthwork (fill) volumes (E_V) are:

$$E_V = \sum_{i=1}^{T} A_i^b \left(Z_{b_i}^{ave} - Z_{g_i}^{ave} \right) \qquad (8)$$

where: A_i^b : base area of cell i, $Z_{g_i}^{ave}$: average ground elevation of cell i, and $Z_{b_i}^{ave}$: average base elevation of cell i. Therefore, additional intersection earthwork costs (C_E^I) are:

$$C_E^I = K_F E_V, \text{ where: } K_F = \text{filling cost per cubic meter}(\$/m^3) \qquad (9)$$

To estimate right-of-way costs by identifying the properties affected by the new intersection design. Jha's (2000) method is employed. Jha divided right-of-way costs into three sub items: (1) temporary easement costs, which are defined as the partial taking of a property during the construction, (2) just compensation costs combining damage, site improvements and cost of the fraction of property taken by the alignment, and (3) appraisal fees. That is,

$$C_R^L = \sum_{i=1}^{n} C_{RW_i} = \sum_{i=1}^{n} (C_{TE_i} + C_{JC_i} + C_{AF_i}) \qquad (10)$$

where: C_{TE_i} = cost of the fraction of property i taken for temporary easement, C_{JC_i} = just compensation paid for property i, C_{AF_i} = appraisal fees for property i. C_{JC_i} is specified as:

$$C_{JC_i} = C_{DP_i} + C_{DS_i} + C_{SI_i} + C_{F_i} \tag{11}$$

where: C_{DP_i} = cost of damage to the value of property i, C_{DS_i} = cost of damage to structures on property i, C_{SI_i} = cost associated with site improvements of property i, and C_{F_i} = cost of the fraction of property i taken for the alignment or intersection.

Generally, the computation takes into account the residual values of properties and pieces of properties left when a given alignment or an intersection is implemented. These values are affected by the size, shape and relative isolation of properties. The estimation procedures largely automate and computerize the existing appraisal process of the Maryland State Highway Administration's Office of Real Estate.

When intersections are added to an alignment, accidents tend to increase since intersections are more hazardous than basic highway segments. Many accident models have been developed to predict frequencies of accidents based on different intersection configurations(Lau and May, 1998; Vogt and Bared, 1998; Sayed and Rodriguez; 1999; Khan et al., 1999).

This study employs two different methods for two representative intersection types on two-lane highways. Lau and May's model (1988) is used for signalized intersections while Vogt and Bared's model (1998) is adopted for two-way stop controlled (TWSC, on minor road) intersections. All-way stop controlled (AWSC) types are excluded since those are less likely to be employed for two-way rural highways.

Lau and May developed an accident prediction model for signalized intersections using the Traffic Surveillance and Analysis System (TASAS) in California. They derived macroscopic-type regression models using millions of vehicles entering intersection for only injury accident models per year but also argued that the models can be used for fatal and property damage only (PDO) accidents based on a grouping and classifying technique called Classification and Regression Trees (CART).

Recently, Vogt and Bared (1998) developed accident prediction models, which used several geometric factors as independent variables, for intersections that are stop-controlled on the minor leg using Minnesota databases obtained from Highway Safety Information System (HSIS) files for the period 1985 to 1989:

Independent variables for applying the above models can be obtained from the geometry of highway alignment alternatives which are generated from the exiting genetic algorithms. Accident frequencies are then calculated using the introduced accident models. After obtaining accident frequencies for intersections, accident costs are calculated by multiplying unit costs, U_c, per accident.

Intersections inherently generate additional delays which should be incorporated into intersection cost functions. For delay cost estimation, this study tries to adopt already developed models since this subject has been studied extensively (Webster, 1958: Robertson, 1969; Wallace et al., 1991; Wallace et al, 1998; FHWA HCM, 1994).

Webster's method to estimate the delays of isolated signalized intersections is adopted since our interest is in rural intersections where oversaturated conditions are rare. For an unsignalized intersection, HCM method is used. Intersection delay costs are finally calculated by introducing unit delay costs, U_s.

Another intersection cost sensitive to alignments is the vehicle fuel cost. Vehicle fuel costs for basic highway segments were already well developed and employed by Jong (1998) based on the average running speed. However, additional fuel costs caused by new intersections were not considered.

There are four types of fuel cost models: (1) Instantaneous models (Akcelik et al., 1983; Bowyer, 1986; Biggs, 1988), (2) Delay type models (FHWA, 1984; Bauer 1975; Courage and Parapar, 1975), (3) Speed type models (Evans et al., 1976; Herman and Ardekani, 1985) and (4) Analytical Models (Liao and Machemehl, 1998). In this study, speed-type and delay-type models are used to build up a new model incorporating Jong's approach. More detailed discussion can be found at Kim et al.(2001).

3.2. Bridge Cost Functions

According to bridge engineering conventions, bridge costs consist of superstructure costs and substructure costs. Those two cost components are functions of number of spans, span lengths, types, materials, pier heights and topography. There is no simple formula for estimating the bridge costs based on all these variables. Only theoretical linear functions for superstructure and substructure costs based on one variable, namely span lengths, are available from the literature (O'Connor, 1971). Those linear functions are based on simply supported composite (steel and concrete) girder bridges. Among many bridge types, since girder bridges have been most commonly employed as highway bridges, this study considers them to be most representative type.

Once the type, material and length of a bridge are set, the most important factors for bridge cost estimation are to determine (1) how many piers (substructures) should be selected and

(2) where those piers should be located. These two factors are interrelated in optimizing bridges.

In bridge engineering, continuity is considered a major factor in optimizing pier locations or span lengths. Continuity considerations favor equal spans or at least gradually varying spans. This study develops a simple approach for estimating bridge costs based on such continuity considerations.

Suppose that we consider a highway bridge which is 100 meters long. To maintain continuity (that is, equal spans), we should divide the100-meter length by some number, say K_b. Suppose we choose a 10-meter, a 25-meter and a 33.3-meter for K_b's. Then, we have different number of piers and equally spaced spans for each K_b.

We can then compute the total bridge costs for each case based on the given number of piers and span lengths. This approach is pursued by numerically searching through different values of K_b until we obtain the optimal span as shown in Fig. 2. There is no guarantee that the bridge cost function is convex. Local optima may exist when a bridge is relatively long and the terrain under it unsmooth.

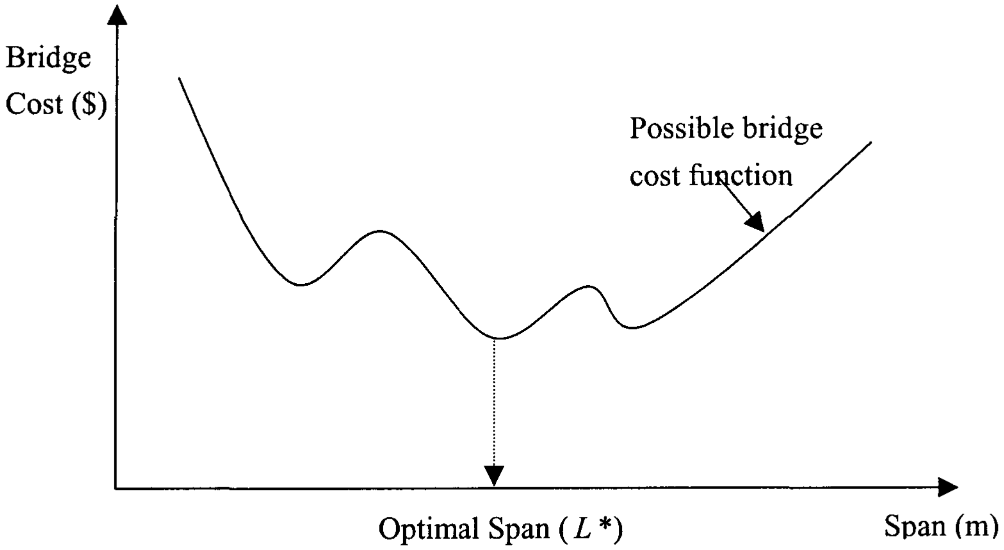


Fig. 2. Graphical representation for getting optimal span of bridges

To estimate bridge costs, already developed linear cost functions for superstructure and substructure developed by O'Connor (1971) are adopted. For the superstructure costs, a linear function (O'Connor, 1971) for superstructure costs (C_U^B) is used.

$$C_U^B = (a_1 + a_2 L) \tag{12}$$

where, L = span length and the coefficients are differentiated by a girder spacing

Eq. (13) explains the substructure costs (C_L^B) for simply supported girder bridges. It shows that costs per foot-width of substructure for variable pier heights increase linearly with span. The coefficients in Eq. (13) differ for different pier heights.

$$C_L^B = (a_3 + a_4 L) \tag{13}$$

3.3. Tunnel Cost Functions

It is reviewed that characteristics affecting small tunnel costs include lengths, cross sections, clearances, horizontal alignments and grades in the previous section. Based on these characteristics and factors, this study formulates (1) earthwork costs and (2) additional costs for small tunnels. Earthwork costs account for lengths, cross sections and clearances, while additional costs account for remaining characteristics and factors.

For estimating earthwork costs, typical desirable cross sections and clearances for a two-lane highway tunnel configuration shown in Fig. 3 are considered. This study refers to two parameters of Fig. 3, the height (h_T, 4.3 m) and the width of the tunnel (w_T, 13.2 m) and only uses the tunnel radius (R, m), since tunnels are usually excavated with circular cross-sections.

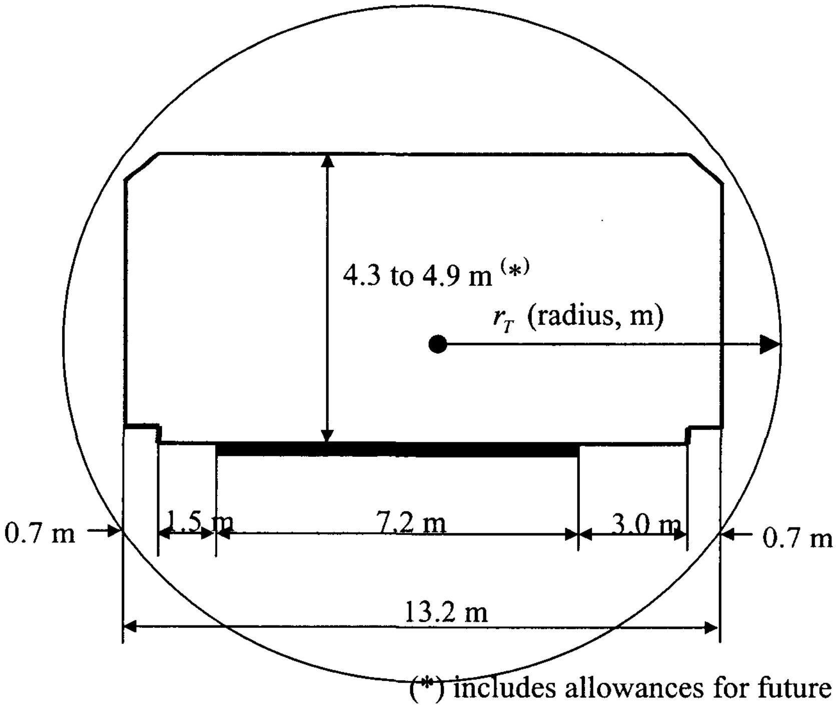


Fig. 3. AASHTO typical desirable cross section and clearances for a two-lane tunnel

Therefore, the tunnel earthwork costs (C_E^T) are:

$$C_E^T = \pi K_T L_T (r_T)^2 \tag{14}$$

where, K_T = tunnel earthwork unit cost per cubic meter ($\$/m^3$) and L_T = tunnel length (m)

The tunnel earthwork cost function, shown in Eq (14), is a linear function of tunnel length. However, the other costs for ventilation, lighting, fire safety, surveillance and traffic controls may not be captured with a linear function of tunnel length. For estimating additional tunnel costs (C_a^T), it is desirable to find functional forms developed from previous studies. Unfortunately, the cost functions for those items are hard to obtain. We may then rely on reliable databases for each affecting factor, from which some functional forms can be developed. This study does not develop the cost functions for those factors. Here, a quadratic function of tunnel length is simply introduced for preliminary analysis. We can easily envision that some factors such as ventilation, lighting, fire life safety provisions and traffic control systems have a more than linear effect on tunnel costs.

$$C_a^T = \alpha_1^T (L^T)^2 + \alpha_2^T (L^T) + \alpha_3^T \tag{15}$$

It is important at this point to remember that the optimization processes for highway alignments should involve both dominating and sensitive cost items. For the preliminary alignment optimization, the effects of tunnel excavation costs may dominate the other cost items such as ventilation, fire safety and surveillance. Also, these additional costs for rural two-lane highways might not be sensitive to highway alignments compared to tunnel excavation costs.

Moreover, for normal operations, naturally ventilated and traffic-induced ventilation systems are considered adequate for relatively short tunnels (less than 180 m (600 ft)) with low traffic volumes (Bendelius, 1996). Also, in most cases, a lighting system is not required inside short tunnels, normally less than 45 m (150 ft) (Mowczan, 1996) and fire safety provisions and traffic control systems may not be deployed for short two-way rural highway tunnels. In summary, this study assumes that tunnel construction costs mainly represent tunnel excavation costs and additional tunnel costs in Eq. (15) are only applied where needed.

3.4. Cost Functions for Grade Separation Structures (Underpass and Overpass)

Grade separation structures might be considered as small bridges. Therefore, bridge cost functions developed in section 3.2 are adopted just for the construction (both superstructure and substructure) costs.

If the elevation difference (Δh) between the ground and a new road satisfies the needed vertical clearance, all cost functions used for a basic highway segment can be employed except for bridge costs. If the elevation difference is not satisfied, additional earthwork and right-of-way costs for providing sufficient vertical and lateral clearances should be estimated. For the additional earthwork costs, let Δh , Δh^+ and Δh^- be elevation differences between a ground elevation and a new road, a needed overpass vertical clearance and a needed underpass vertical clearance, respectively.

Suppose that the vertical clearance is not satisfied (i.e., $\Delta h^- < \Delta h < \Delta h^+$). Then, additional earthwork costs are a function of distances between stations (Δs) and $|\Delta h^+ - \Delta h|$ (for overpass case) or $|\Delta h^- - \Delta h|$ (for underpass case).

Fig. 4 shows a typical fill cross section where the road elevation should be raised by Δh^{+}. Then, the additional cross sectional area (ΔA) is:

$$\begin{aligned}\Delta A &= W\varepsilon^{+} + \left(\Delta h^{+}\cot\theta_1 - \Delta h\cot\theta_1\right) + \left(\Delta h^{+}\cot\theta_2 - \Delta h\cot\theta_2\right) \\ &= \varepsilon^{+}\left(W + \cot\theta_1 + \cot\theta_2\right)\end{aligned} \tag{16}$$

Therefore, the additional earthwork volumes (ΔV) and costs (ΔC_E) are:

$$\Delta V = \varepsilon^{+}\left(W + \cot\theta_1 + \cot\theta_2\right)\Delta s \tag{17}$$

$$\Delta C_E = K_F\varepsilon^{+}\left(W + \cot\theta_1 + \cot\theta_2\right)\Delta s \tag{18}$$

where: K_F = unit filling cost per cubic yard.

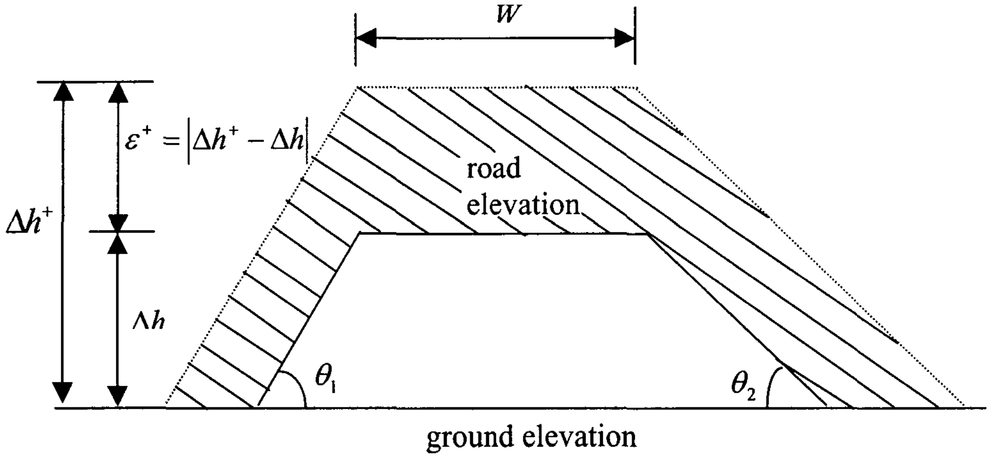


Fig. 4. Typical cross section with insufficient vertical clearances

Likewise, an additional right-of-way cost function might be formulated as a function of $|\Delta h^{+} - \Delta h|$ for an overpass or $|\Delta h^{-} - \Delta h|$ for an underpass. Intuitively, assuming relatively flat topography, the area affected by raising the road elevation by ε^{+} is a quadratic function of ε^{+}.

$$\Delta C_R = \alpha_1^R \left(\varepsilon^+\right)^2 + \alpha_2^R \varepsilon^+ + \alpha_3^R \tag{19}$$

In Eq. (19), the values of each coefficient depend on topography, associated side slope (θ), land use, sizes and shapes of associated parcels and other factors.

3.5. Interchange cost functions

As specified in the research scope, cost functions are formulated here for three types of interchanges: (1) diamond, (2) clover and (3) trumpet types. Diamond and clover types are for four-leg interchanges while trumpet types are for three-leg interchanges.

Interchange cost functions are basically compilations of already developed cost functions for other structures. For instance, Fig. 5 shows centerlines of the associated roads and ramps for a diamond interchange and two major design criteria to consider: (1) major interchange leg length, l_d^b and (2) minor interchange leg length, l_d^s.

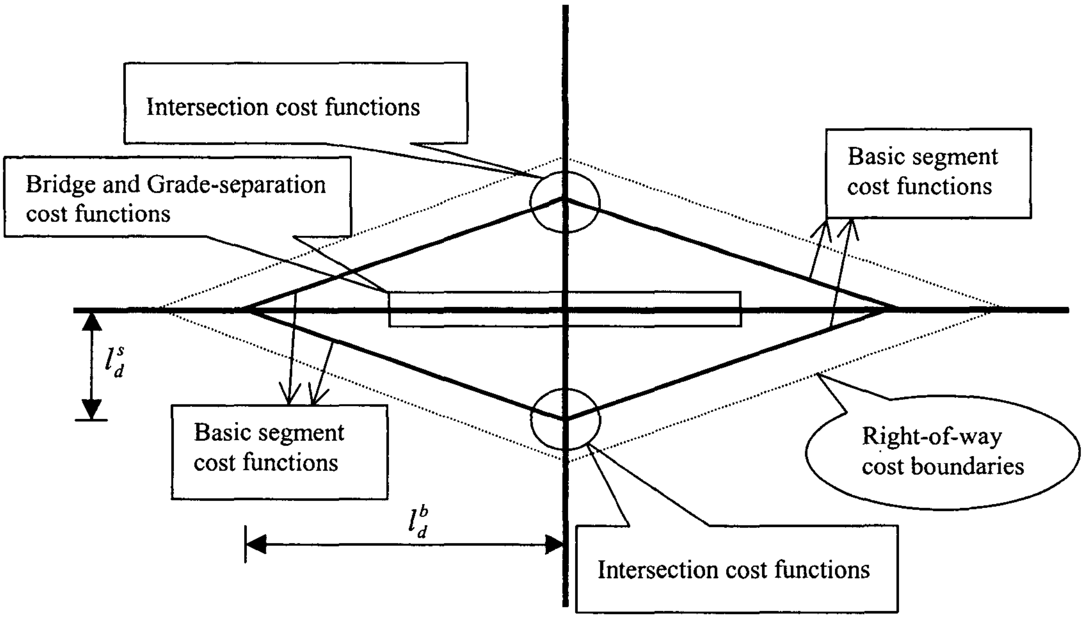


Fig. 5. Centerlines of a typical diamond interchange, major design criteria and separated parts for each cost function

The diamond interchange is divided into several parts to be evaluated using already developed formulations for intersections, basic segments, bridges and grade separations. Similarly, clover interchange costs can be obtained. Fig. 6 shows the centerlines of a clover interchange. Three major design criteria are important in evaluating the total costs: (1) interchange leg length, l_c, (2) outside turning ramp radius (r_c^o) and (3) inside turning ramp radius (r_c^i). The total costs can be evaluated by employing the necessary cost functions for each structure.

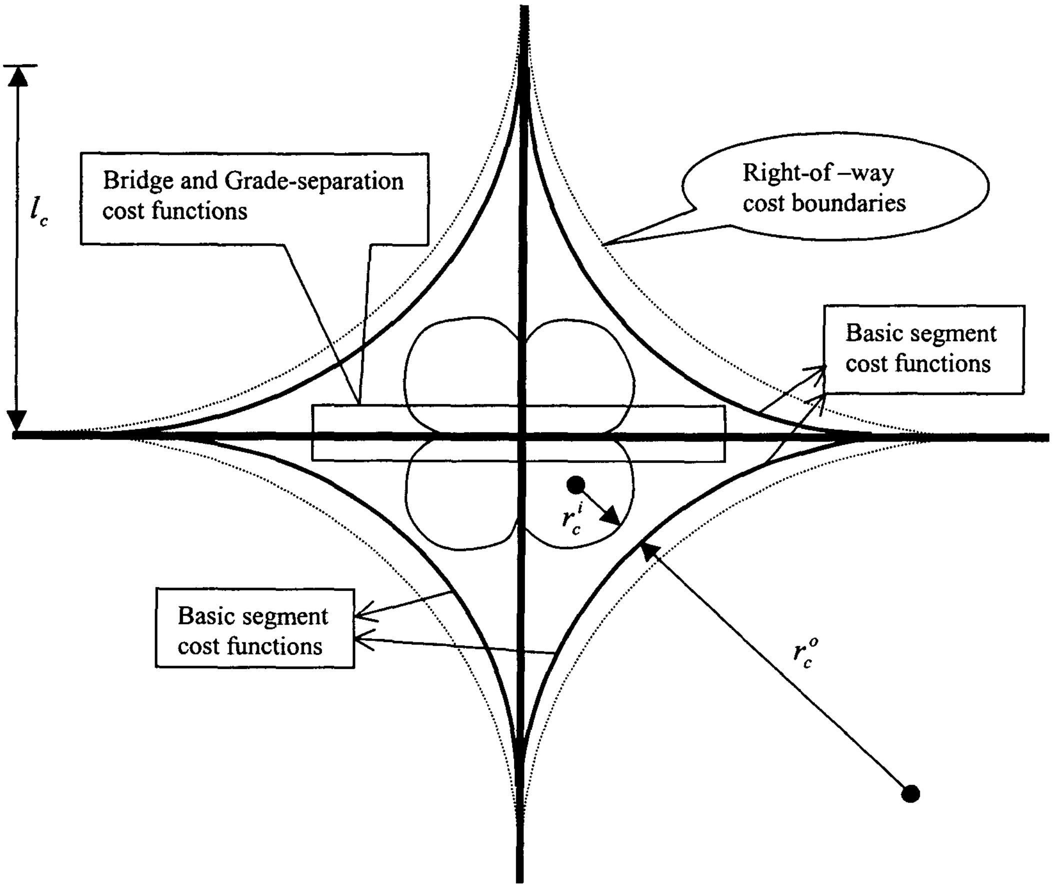


Fig 6. Centerlines of a typical clover interchange, major design criteria and separated parts for each cost function

For trumpet interchanges, the same procedure is also adopted. In it two major design criteria should be considered: (1) outside turning ramp radius (r_t^o) and (2) inside turning ramp radius (r_t^i).

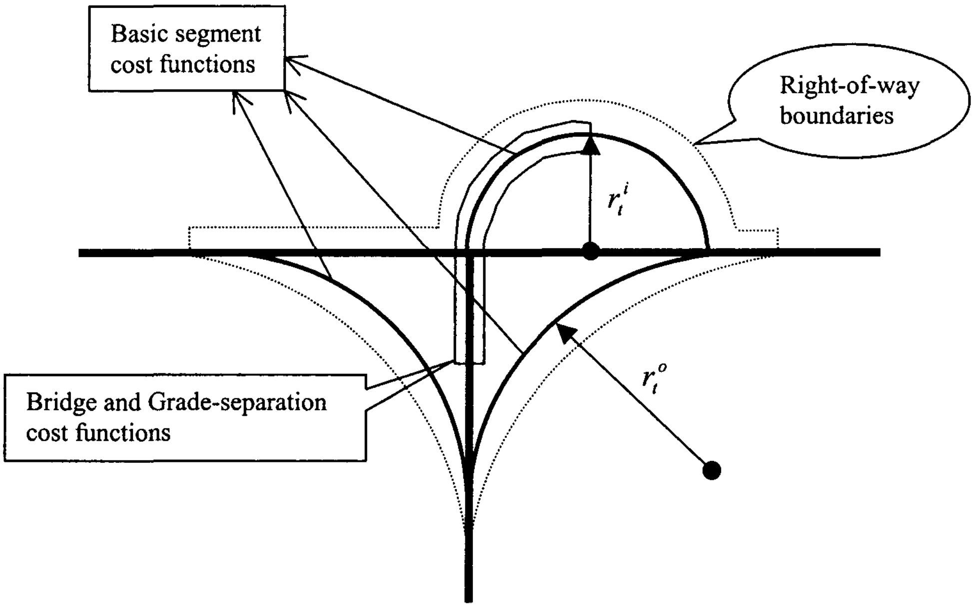


Fig. 7. Centerlines of a typical trumpet interchange, major design criteria and separated parts for each cost function

4. LOCAL OPTIMIZATION OF INTERSECTION

When an intersection is considered as a crossing type between a new alignment and an existing road, the crossing angle is a very important constraint to satisfy. If crossing angles are overly acute, a new alignment has been discarded.

However, the new alignment crossing with an acute angle is superior to other alternatives and that discarding it simply because of the intersection angle might be undesirable overall. It might be desirable to pursue a method that could perturb the local geometry to produce a better intersection, yet retain the broader geometry of the good candidate alignment. Fig. 8 shows an example of how a better solution might be obtained.

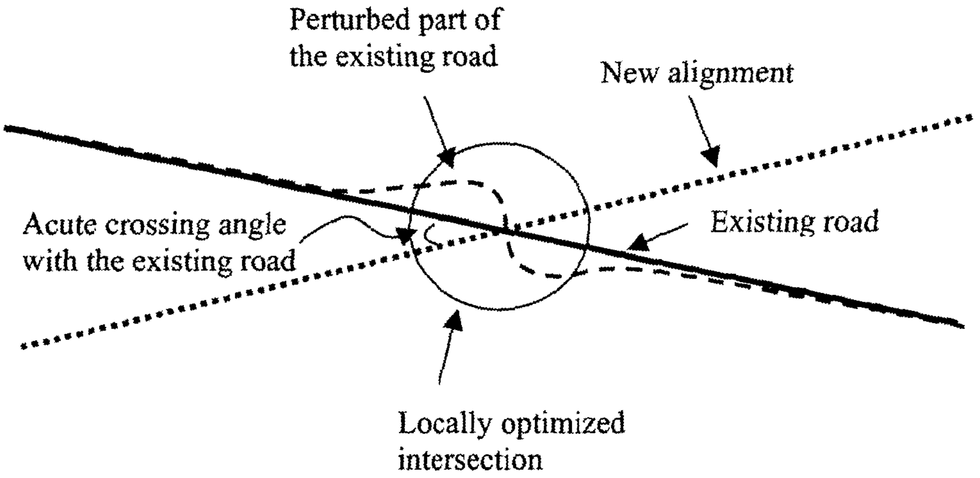


Fig. 8. A Locally optimized intersection for an intersection with an acute angel

4.1. Mathematical Expressions for a Perturbed intersection

To describe highway alignments, a parametric representation is useful. Boldface capital letters will be used to denote vectors in space. It is assumed that the local optimization process described herein resides within a larger alignment optimization framework. For local optimization to take place, it must be the case that an alignment alternative has been generated that crosses an existing road at an unacceptable angle, θ, as described earlier. The existing roadway presumably is described in a databases (for example, GIS), and the most common form would be piecewise linear, with points $\{\mathbf{E}_i\}$ representing the segment endpoints. The proposed new alignment can be described similarly, although we adopt a form more common in highway design, consisting of a sequence of tangent sections and circular arcs. We assume that station points, $\{\mathbf{D}_i\}$ are defined along this alignment at regular intervals specified by the user.

The series of station points in the vicinity of the proposed intersection constitutes the domain of our decision variable, which is the location of the newly aligned intersection. The question of what constitutes the "vicinity" is up to the model user. On either side of the proposed intersection, $\mathbf{I}$, we consider at least one of the existing roadway nodes, $\{\mathbf{E}_i\}$. These need not fall within our vicinity. However, if several of them happen to do so, then they all must be considered. The decision variable $\mathbf{D}$ represents the potential location of the intersection.

Suppose that a new alignment has been developed, and represents the major road at the intersection with an existing road. The existing minor road, therefore, will need to be perturbed. Fig. 9 shows a general alternative where intersection legs are longer. Since $\mathbf{P}_1$ is between $\mathbf{E}_1$ and $\mathbf{E}_2$, we need to know θ'. It can be obtained as follows:

$$\theta' = \cos^{-1}\left(\frac{(\mathbf{E}_2 - \mathbf{E}_1)\cdot(\mathbf{I} - \mathbf{D})}{\|\mathbf{E}_2 - \mathbf{E}_1\|\|\mathbf{I} - \mathbf{D}\|}\right) \tag{20}$$

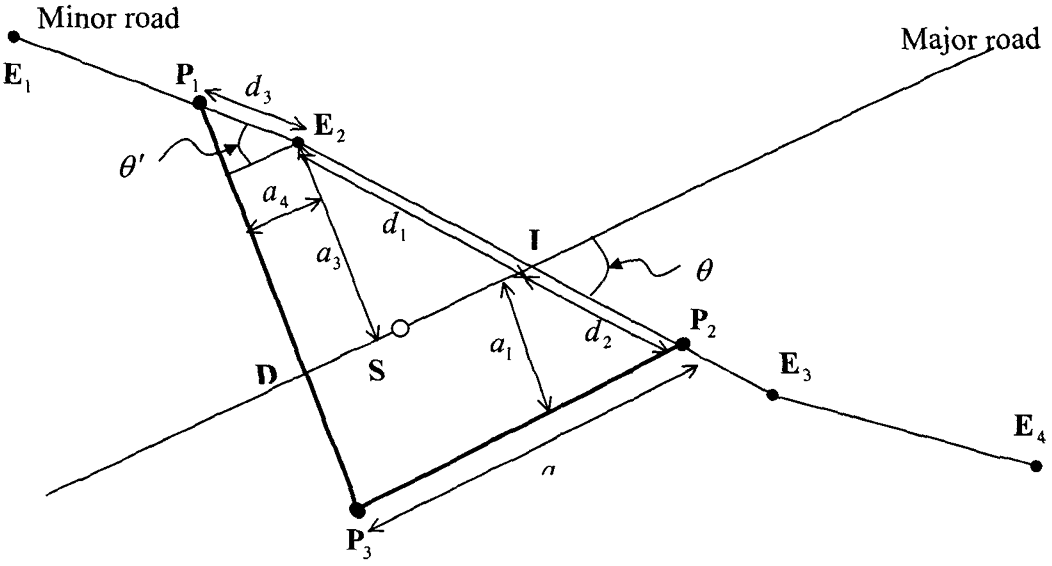


Fig. 9. Interesting points for a general alternative of local intersection optimization

Then, using $a_3 = d_1 \sin\theta$, $a_4 = \|\mathbf{I} - \mathbf{D}\| - \sqrt{(d_1)^2 - (a_3)^2}$, $d_3 = \frac{a_4}{\cos\theta'}$ and $d_2 = \frac{a_1}{\cos(90-\theta)}$, all the remaining coordinates can be obtained as follows:

$$\mathbf{P}_1 = \mathbf{E}_2 + \left(\frac{\mathbf{E}_2 - \mathbf{E}_1}{\|\mathbf{E}_2 - \mathbf{E}_1\|}\right)(-d_3),\ \mathbf{P}_2 = \mathbf{I} + \left(\frac{\mathbf{I} - \mathbf{E}_2}{\|\mathbf{I} - \mathbf{E}_2\|}\right)(d_2)$$

$$\text{and } \mathbf{P}_3 = \mathbf{D} + \left(\frac{\mathbf{D} - \mathbf{P}_1}{\|\mathbf{D} - \mathbf{P}_1\|}\right)(a_1) \tag{21}$$

Based on these coordinates, any point on the newly evaluated intersection legs can be obtained. This helps formulate each cost item by easily identifying where the legs and the crossing point (intersection) are located within a study area.

5. CASE STUDIES

In this section the developed cost functions and algorithms for modeling intersections and other structures are all incorporated together into highway alignment optimization. One fairly complex artificial study area and one real GIS application in Washington Country, Maryland, are employed to test applicability of the developed methods. Fig. 10 shows the quite complex topography of the artificial study area which includes a two-lane rural highway from the center of North to South East, three hills and a creek crossing from North East edge to South. Our plan is to build a two-lane rural highway connecting two specified end points while allowing the existing road to be re-optimized.

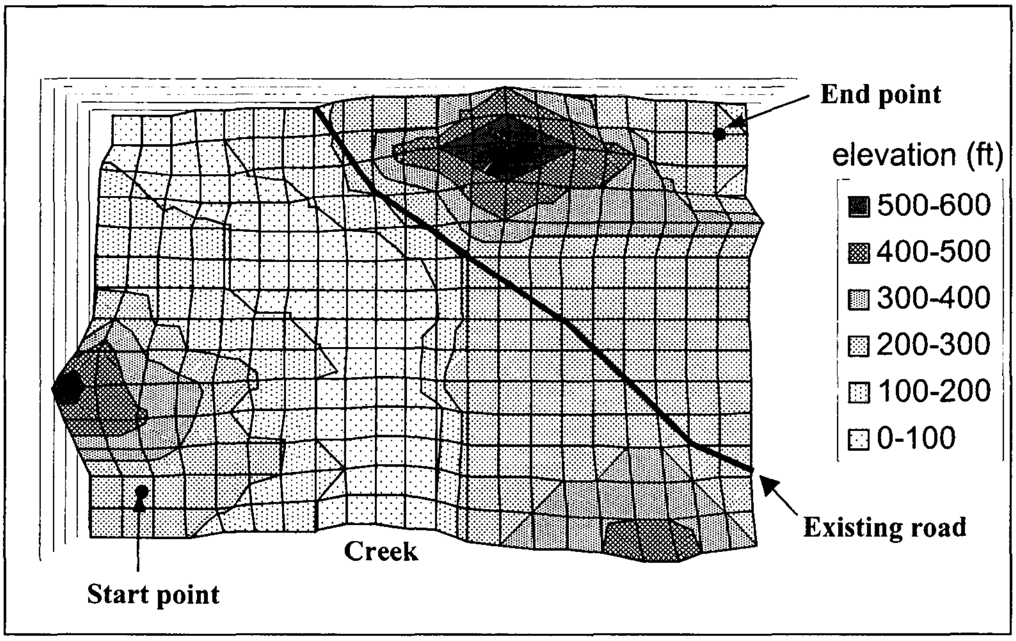


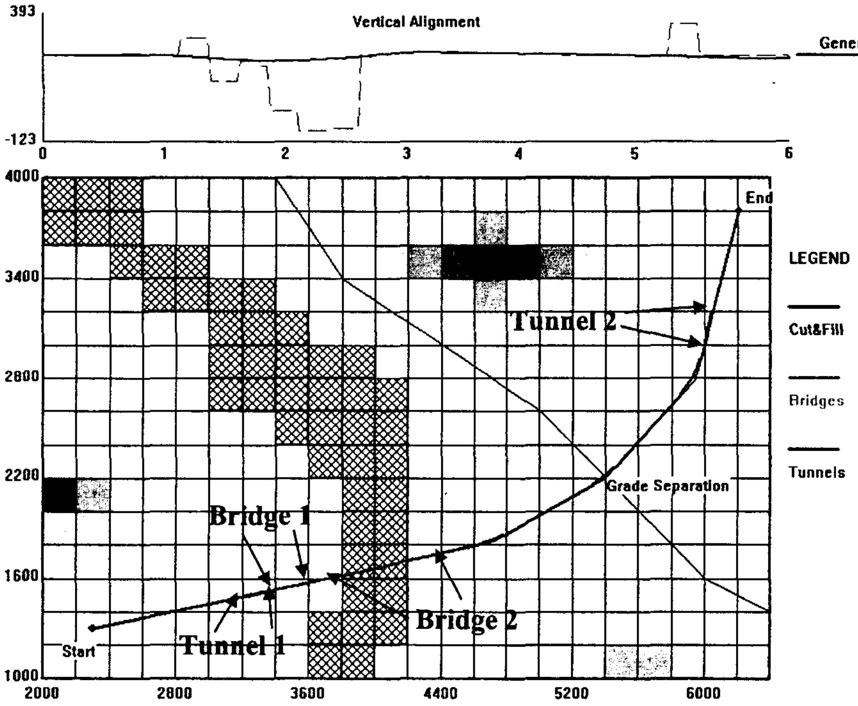
Fig. 10. Topography of the Artificial Study Area

Three types of test scenarios in Table 2 are designed to examine how the methods work under various situations. Since the model is designed to automatically select the minimum cost crossing type of the new alignment with the existing road, none of the crossing type is specified for the first test run (scenario 1). In the second and third scenarios, it is assumed that users specify an intersection and an interchange, respectively, as the crossing type with the existing road. A desktop with 1 GHz CPU speed and 261 MB RAM is used to run the program.

Table 2 Scenarios for modeling intersections and other structures

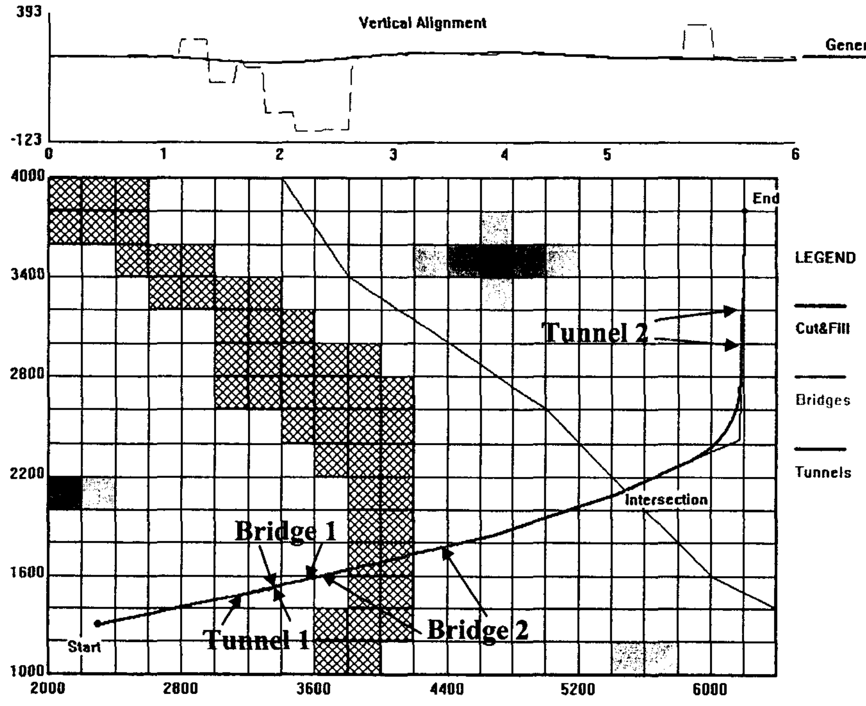
Scenarios	User specified crossing type with the existing road	No. of generations	Local optimization of intersections
Scenarios 1	None	500	Yes, if initiated
Scenarios 2	Intersection	500	Yes, if initiated
Scenarios 3	Interchange(diamond)	500	No

Figs. 11, 12 and 13 show the optimized solutions under scenarios 1, 2 and 3. For scenarios 1, grade separation is selected on the minimum cost crossing type with the exiting road and the best solution has two bridges and two tunnels. For scenarios 2, local intersection optimization is not initiated for the best solution obtained by the algorithm since the crossing angle is approximately 70 degrees. The best solution for the scenario 2 also contains two bridges and tunnels at similar location to scenario 1. For scenario 3, a similar solution, but involving only one bridge, is found with a diamond interchange.



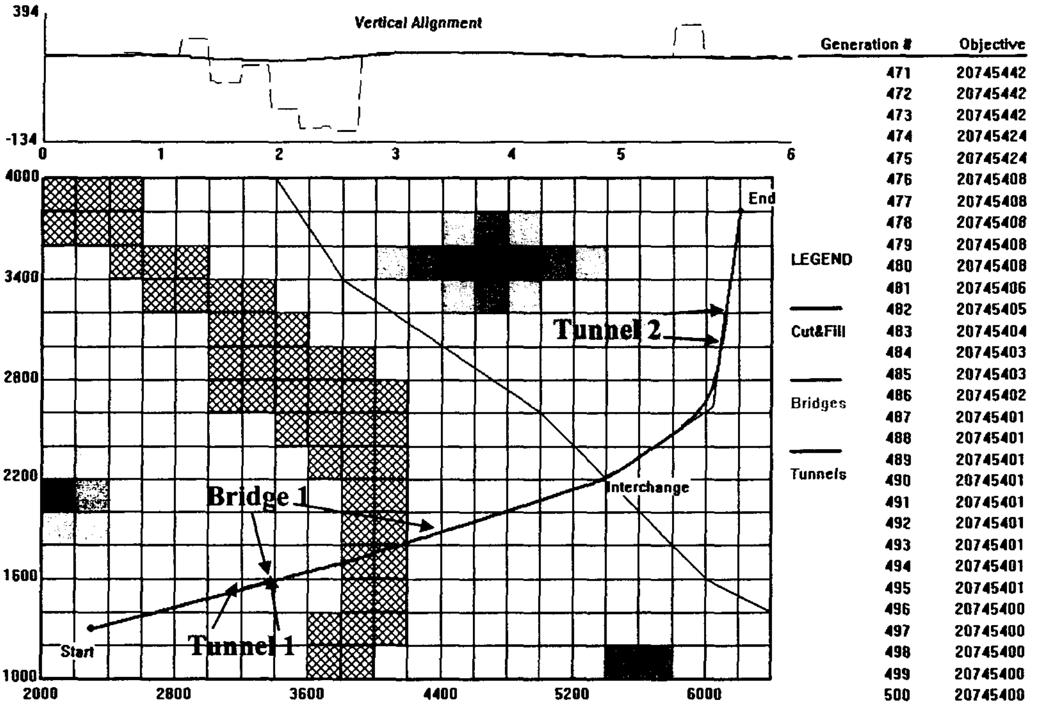
Generation #	Objective
471	18826952
472	18815345
473	18815342
474	18815342
475	18815342
476	18815342
477	18815342
478	18815341
479	18815341
480	18815341
481	18815339
482	18815339
483	18815339
484	18815337
485	18815337
486	18815337
487	18815336
488	18815336
489	18815336
490	18815336
491	18815336
492	18815336
493	18815336
494	18815336
495	18815336
496	18815336
497	18815336
498	18815336
499	18815336
500	18815336

Fig. 11. Solution for Scenario 1



Generation #	Objective
471	21035755
472	21035638
473	21035638
474	21035638
475	21035527
476	21035527
477	21035518
478	21035350
479	21035350
480	21035350
481	21035200
482	21035185
483	21035077
484	21034930
485	21034882
486	21034787
487	21034787
488	21034696
489	21034655
490	21034423
491	21034035
492	21034035
493	21033760
494	21033760
495	21033374
496	21033374
497	21033374
498	21033302
499	21032886
500	21032874

Fig. 12. Solution for Scenario 2



Generation #	Objective
471	20745442
472	20745442
473	20745442
474	20745424
475	20745424
476	20745408
477	20745408
478	20745408
479	20745408
480	20745408
481	20745406
482	20745405
483	20745404
484	20745403
485	20745403
486	20745402
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490	20745401
491	20745401
492	20745401
493	20745401
494	20745401
495	20745401
496	20745400
497	20745400
498	20745400
499	20745400
500	20745400

Fig. 13. Solution for Scenario 3

Table 3 provides general information for each scenario. Computation time for scenario 2 (4 minutes 50 seconds) was longer than for scenarios 1 (3 minutes 24 seconds) and 3 (3 minutes 25 seconds) since it uses an additional module for local intersection optimization.

Table 3 General Information for three scenarios

Scenarios	Best generation	Total Cost (\$)	Computation time	Crossing		No. of tunnels	No. of bridges
				Type	Costs (\$)		
Scenario 1	500	18.82 million	3 minutes 24 seconds	Grade Separation	91,260	2	2
Scenario 2	500	21.03 million	4 minutes 50 seconds	Intersection	1.49 million	2	2
Scenario 3	499	20.75 million	3 minutes 25 seconds	Diamond interchange	1.19 million	2	1

Table 4 shows detailed cost breakdowns for three scenarios.

Table 4 Cost breakdowns for three scenarios

	Scenario 1	Scenario 2	Scenario 3
Total costs	18,815,336 (100.00)	21,032,874 (100.00)	20,745,408 (100.00)
Structures	91,260 (0.49) grade separation	1,488,056 (7.07) intersection	1,185,976 (5.72) diamond interchange
Pavement	1,850,426 (9.83)	1,944,525 (9.25)	1,872,715 (9.03)
Right-of-way	4,436,439 (23.58)	4,583,726 (21.79)	4,877,867 (23.51)
Vehicle operation	909,814 (4.84)	957,023 (4.55)	921,015 (4.44)
User time value	5,320,836 (28.28)	5,685,093 (27.03)	5,409,726 (26.08)
Accident	241,482 (1.28)	195,153 (0.93)	147,109 (0.71)
Earthwork	1,554,727 (8.26)	1,722,632 (8.19)	1,674,379 (8.07)
Bridges	2,064,630 (10.97)	2,110,944 (10.04)	2,310,899 (11.14)
Tunnels	2,345,722 (12.47)	2,345,722 (11.15)	2,345,722 (12.31)

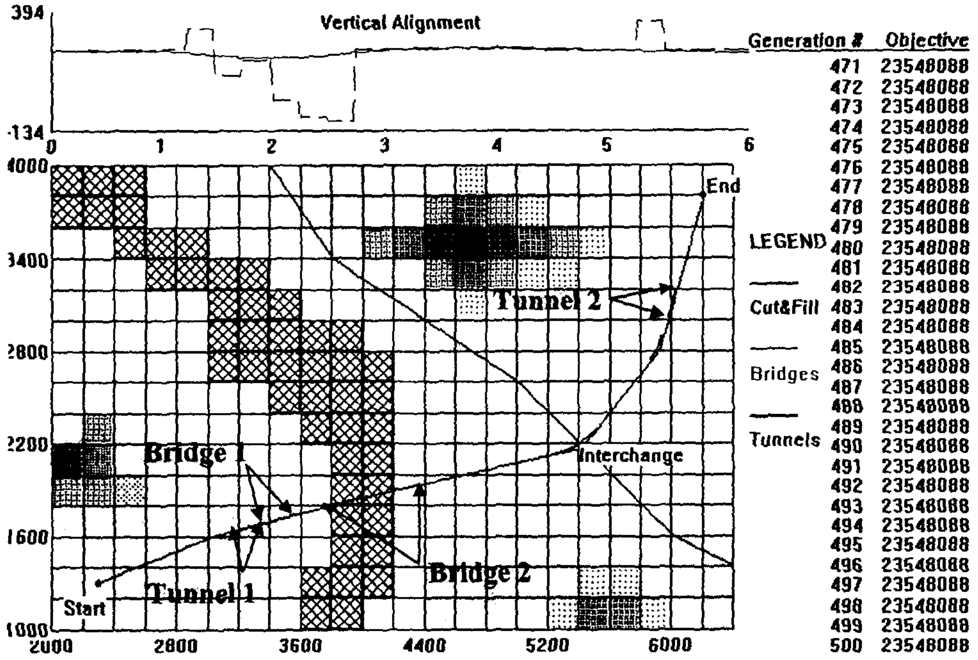
Tunnel costs for the three scenarios are similar since the lengths and locations of the two tunnels are almost the same. Interestingly, the intersection costs of scenario 2 exceed those of diamond interchange for scenario 3. The reasons are clear when we see the detailed cost fractions for the intersection and the interchange as shown in Table 5. The intersection of scenario 2 has more accidents, delay and fuel costs than the interchange.

Table 5 Fractions for the intersection of scenario 2 and interchange of scenario 3

	Intersection of scenario 2	Interchange of scenario 3
Total costs	1,488,056 (100.00)	1,185,976 (100.00)
Pavement	11,809 (0.79)	9,928 (0.84)
Earthwork	34,625 (2.33)	342,396 (28.87)
Right-of-way	279,953 (18.81)	559,724 (47.20)
Additional delay	509,549 (34.24)	0
Additional fuel	27,415 (1.84)	0
Additional accident	624,705 (41.98)	0
Structure (grade separation)	0	91,260 (7.69)
Additional intersection	0	182,668 (15.40)

It is desirable at this point to check how the optimized spans for bridges are found. To preserve the continuity of spans, integer numbers of piers with real number span lengths are analyzed. Also, the high water level flooding constraint ($\varepsilon_w = 3.3\text{m}$) is applied. The study area from scenario 3 is used and a diamond interchange is specified for the crossing type with the existing road.

Fig. 14 shows the optimized solution involving two bridges and two tunnels. Bridge is 54 m (180 ft) long and bridge 2 is 174 m (580 ft) long. Bridge 2 is then analyzed for the optimized span. Table 6 and Fig. 15 illustrate how the optimized span is obtained with integer numbers of piers. Ten span lengths (6.91 m through 87 m) are analyzed, using integer number of spans from 25 to 2. The optimized span is obtained as 29.01 m, with 5 piers.



Generation #	Objective
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473	23548088
474	23548088
475	23548088
476	23548088
477	23548088
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489	23548088
490	23548088
491	23548088
492	23548088
493	23548088
494	23548088
495	23548088
496	23548088
497	23548088
498	23548088
499	23548088
500	23548088

Fig. 14. Optimized Solution, Considering a High Water Constraint and Integer Numbers of Piers

Table 6 Optimized span and cost breakdown for bridge 2, considering a high water constraint and integer numbers of piers

No.of spans	Span (ft)	Total bridge costs ($)	Substructure costs ($)	Superstructure costs ($)	Optimized span (ft)	Minimum bridge costs ($)
25	23.2	3,281,835	2,658,577	623,258	**96.7**	**1,359,190**
20	29.0	2,709,342	2,074,619	634,723		
15	38.7	2,088,050	1,478,341	609,709		
10	58	1,772,312	1,011,664	760,648		
8	72.5	1,732,725	862,955	869,770		
7	82.9	1,505,560	638,570	866,990		
6	**96.7**	**1,359,190**	**490,708**	**868,482**		
5	116.0	1,592,510	449,513	1,142,997		
4	145.0	1,753,063	344,941	1,408,122		
2	290.0	2,761,115	157,868	2,603,247		

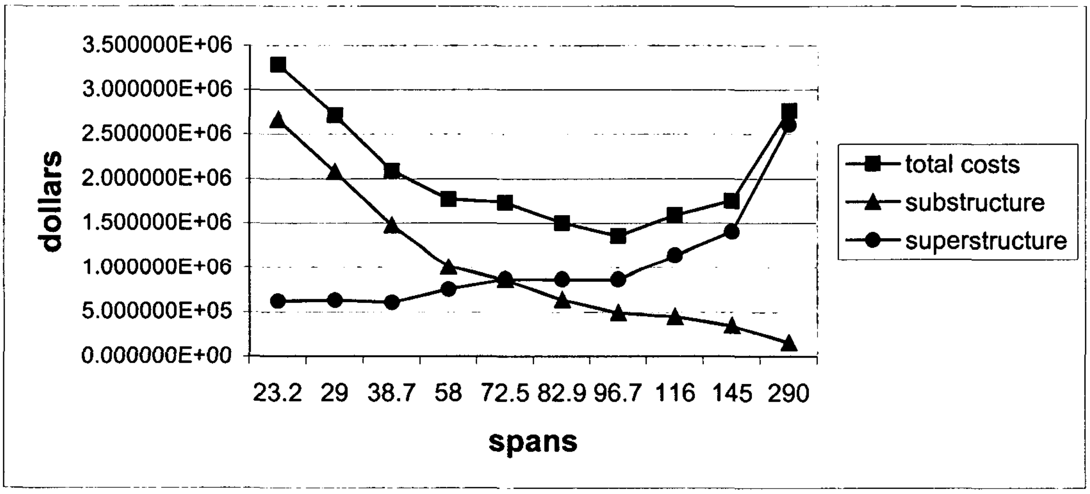


Fig. 15. Optimized span and cost breakdown for bridge 2, considering a high water constraint and integer numbers of piers

Until now, the optimized solutions did not involve local intersection optimization. To check how local intersection optimization affects the solutions, the original start and end points are deliberately moved, as shown in Fig. 16. After this change, a new solution is obtained in which local optimization is applied, as shown in Fig. 17. The new solution is obtained at generation 500 and total costs and computation time are found to be $23.54 million, 33 minutes and 54 seconds, respectively. Two tunnels and one bridge are in the new solution and the highway starts with a tunnel.

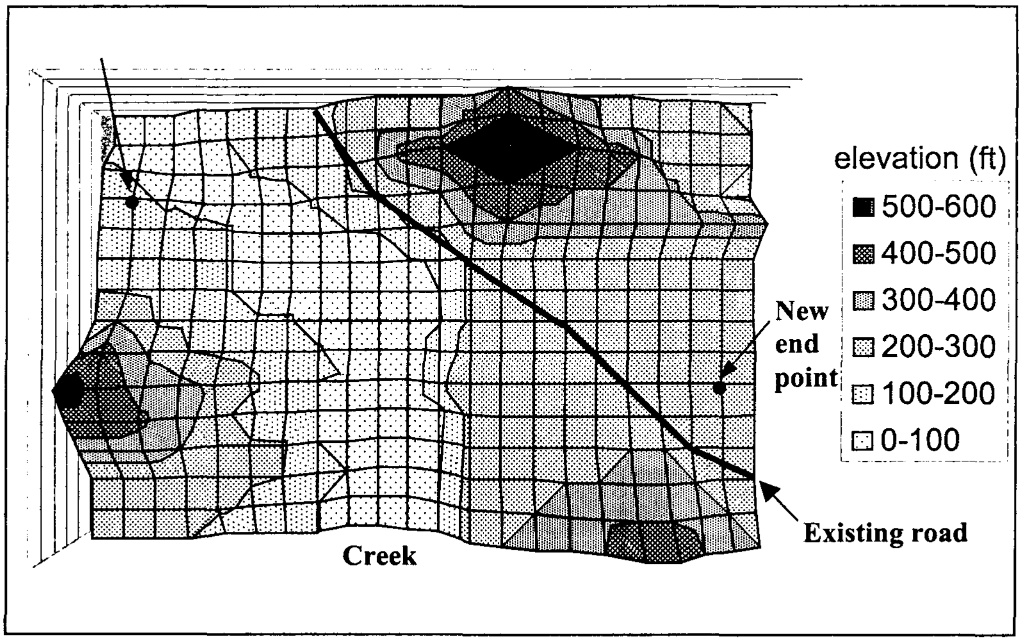
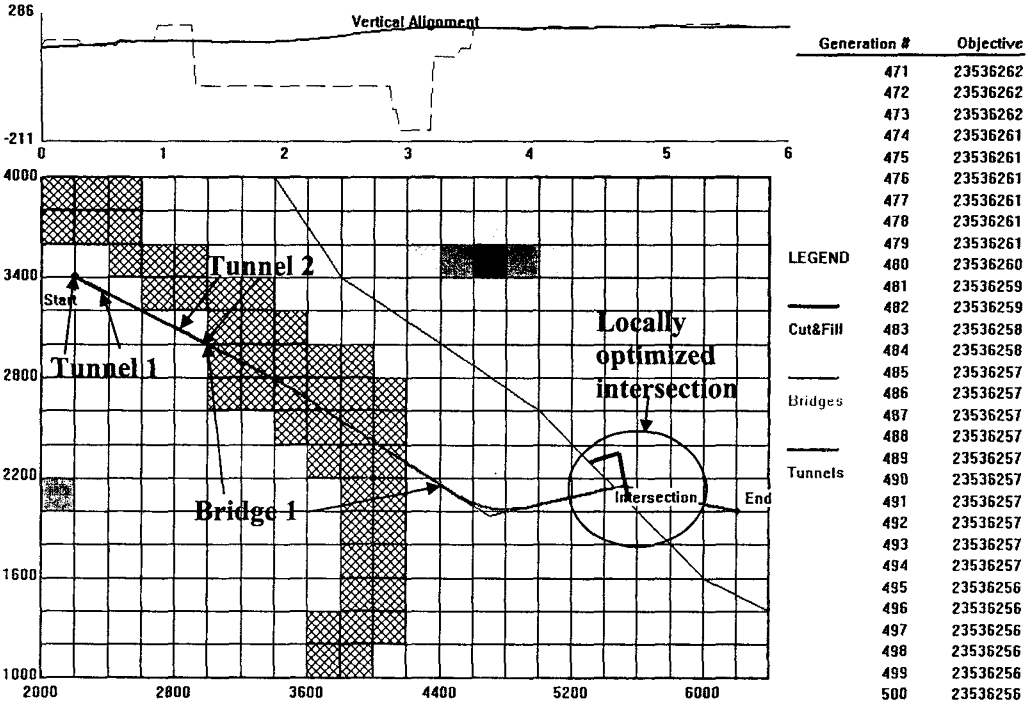


Fig. 16. Deliberately moved start and end points for new example



Generation #	Objective
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474	23536261
475	23536261
476	23536261
477	23536261
478	23536261
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487	23536257
488	23536257
489	23536257
490	23536257
491	23536257
492	23536257
493	23536257
494	23536257
495	23536256
496	23536256
497	23536256
498	23536256
499	23536256
500	23536256

Fig. 17. The optimized solution involving local intersection optimization with moved start and end points

The crossing angle is approximately 48 degrees. With this local optimization of intersections, it is observed that approximately $\$10^6$ are saved. (The estimated original intersection costs were \$3.61 million and the newly perturbed intersection costs were \$2.59 million.) More importantly, the best solution found is not discarded during successive generations just because of the unacceptable crossing angle between the existing road and the new alignment.

Another case study with a real GIS map from Washington Country, Maryland is conducted to check how the developed cost functions and algorithms for intersections and other structures can be integrated with a GIS. The study area is relatively large and quite complex compared to the previous case examples. It contains 416 properties (306 residential, 75 agricultural and 12 commercial) including highways and creeks. Concoheague creek is winding in the upper and left parts of the study area and there are four major highways : MD 494 crossing the study area from West to East, MD 58 at the right top. Broadfording Road at the left bottom and Shinham Road parallel with the creek at the top. Its area is approximately 1.02 km^2. Unit land costs range from $\$0.11/m^2$ to $\$33.3/m^2$ and costs of structures range from \$ 2,880 to \$ 767,300 per property. Fig. 18 shows a photograph from the web describing a relatively flat study area.

Fig. 18. Aerial photo of the study area, Washington County, Maryland
Source: http://terraserver.homeadvisor.msn.com

It is assumed that a connection between Broadfording Road and MD 58 is considered to provide a better access with residents of the study area. This requires a new intersection between the new alignment and MD 494. After all information is combined and 100 generations are run, an optimized solution (Fig. 19) involving local intersection optimization is obtained at generation 99.

The total costs are $ 8.11 million and computation time is approximately 19 hours and 7 minutes. The much longer computation times than those observed in artificial case studies resulted from many factors such as (1) communication between two different computing environment, GIS and C software, (2) local intersection optimization during the program run, (3) many alternatives causing local intersection optimization through the successive generations, (4) a relatively large number of properties in the study area and (5) more generations, i.e., 100.

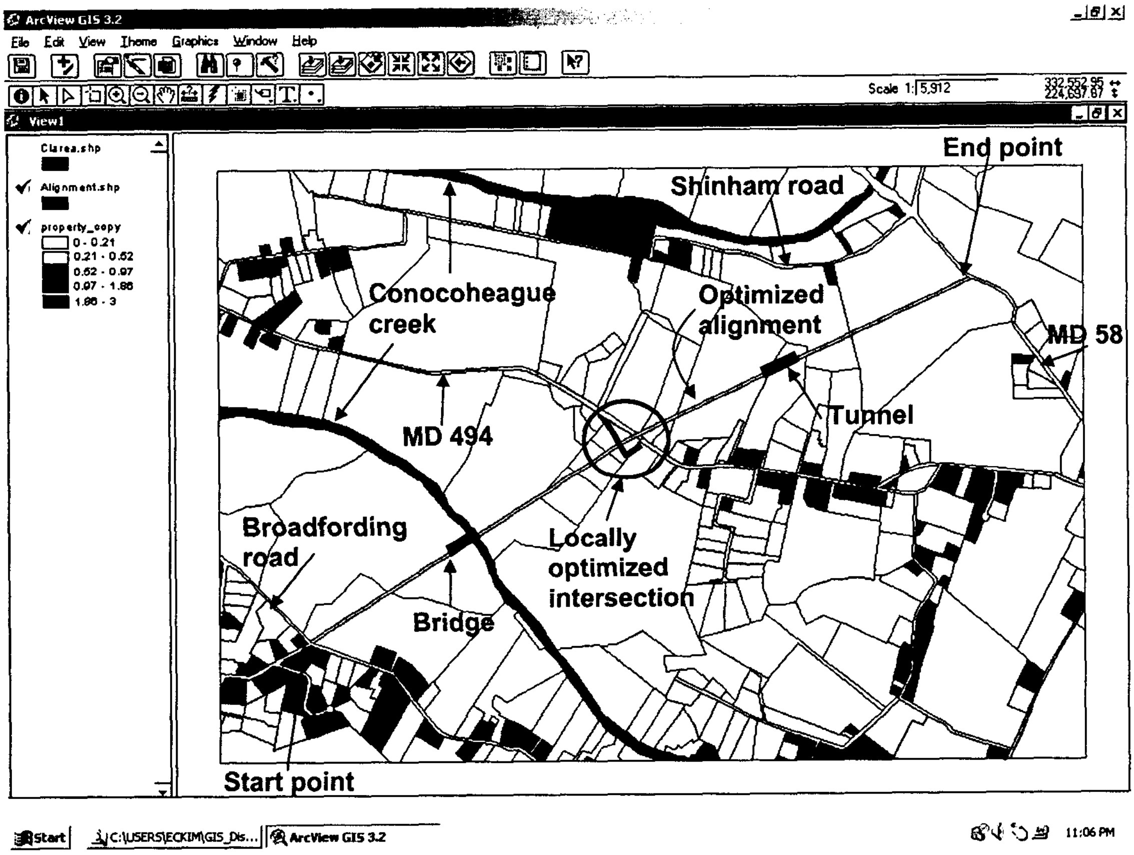

Fig. 19. Optimized solution for the study area, Washington County, Maryland

The optimized solution is relatively straight. It has one bridge crossing Conocoheague creek and one tunnel. The crossing angle with the existing MD 494 is approximately 58 degrees.

As in the artificial case earlier, approximately $0.9 million are saved with local intersection optimization (see Table 7).

Table 7 Cost comparison between original and locally optimized intersections for Washington Country

Original intersection costs		Local optimization costs	
Cost items	Costs ($) and fractions(%)	Cost items	Costs ($) and fractions(%)
Total costs	1,991,359 (100.00)	Total costs	1,089,186 (100.00)
Pavement	11,809 (0.59)	Link earthwork	35 (0.00)
Earthwork	0 (0.00)	Link right-of-way	69,650 (6.39)
Right-of-way	62,789 (3.15)	Link pavement	13,060 (1.20)
Delay	509,549 (25.59)	New intersection earthwork	0 (0.00)
Fuel	27,415 (1.38)	New intersection right-of-way	37,095 (3.41)
Accidents	1,379,797 (69.29)	New intersection delay	254,775 (23.39)
		New intersection fuel	24,672 (2.27)
		New intersection accident	689,899 (63.34)

The final cost breakdown for this case is shown in Table 8. User costs account for more than 50 % of the total costs. Bridge and tunnels costs are 3.29 % and 6.89 %, respectively. The costs for local intersection optimization are found to be $ 1.09 million and account for 13.43 % of the total costs. It should be remembered that the optimized solution cannot be obtained using the previous methods which did not incorporate local intersection optimization.

Table 8 Final cost breakdown for the solution of Washington Country after local optimization

Cost items	Costs ($) and fractions (%)
Total costs	8,109,296 (100.00)
Local intersection optimization	1,089,186 (13.43)
Pavement	1,280,939 (15.80)
Right-of-way	516,505 (6.37)
Vehicle operation	627,861 (7.74)
User time value	3,403,653 (41.97)
Accidents	47,897 (0.59)
Tunnels	558,505 (6.89)
Bridges	266,927 (3.29)
Earthwork	317,823 (3.92)

In the examples in which local intersection optimization is not employed, it was normal that objection functions decrease very rapidly in the early generations and very slightly near the end. However, when a solution involves local intersection optimization, it is conceivable that the objective function value may drop suddenly because the solution is not obtained from the fine-tuning process of the best solution found at the previous generation. Since the solution obtained involves local intersection optimization, it is presumable that significant changes in the objective function may occur in late generations as well as early Washington Country case study where sudden drops of objective function values occur at relatively later generations.

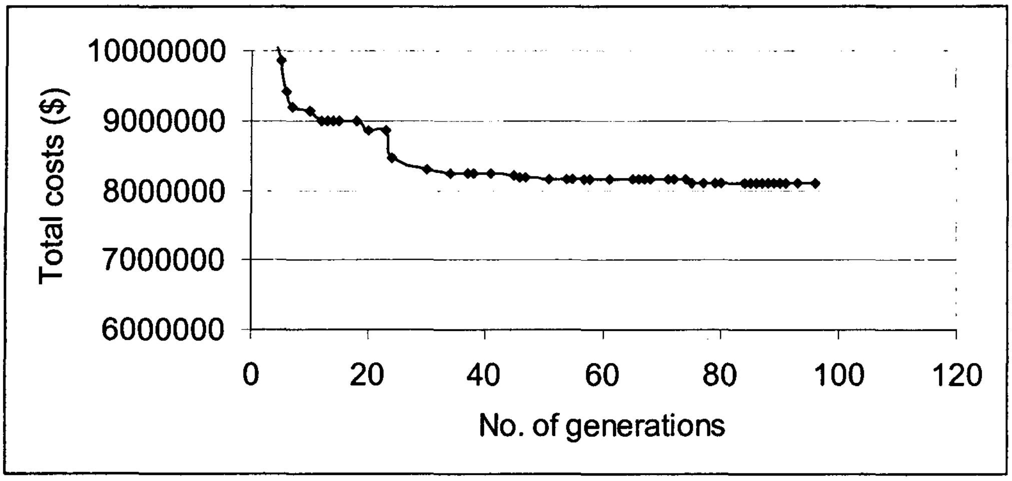

Fig. 20. Changes of objective functions for Washington County case study

6. CONCLUSIONS

The modeling methods for intersections and other structures in highway alignment optimization are developed in this study. Highway alignment optimization is a very complex and challenging problem. The complexity of this problem stems from many cost factors and design constraints to be met. Among them, modeling such important elements of highway as intersections and other structures has been neglected.

In this study, cost function for intersections and other structures are formulated and incorporated into existing models using developed algorithms for each structure. Although the developed cost functions for structures are not sufficient for detailed micro-level design, it is observed that those functions work well enough for alignment optimization preliminary design. It is also assessed that the solutions obtained from several case studies conducted in preceding sections are satisfactory.

The proposed modeling method for intersections is more suitable than other, more laborious, methods for estimating intersection costs, which would not be adaptable in an automated optimal search process. Moreover, the developed method can be used without any optimization process for just evaluating intersection alternatives.

Other structures modeled include bridges, tunnels, grade separations and interchanges. Cost functions for these structures are suitably formulated and successfully incorporated into genetic algorithms in connection with GIS. It is notable that locations of solutions obtained through various case studies are significantly changed compared to the ones found by the existing models, while improving the objective functions.

This study also developed a method to locally optimize intersections within the context of highway alignment optimization. With this method, we can avoid wastefully discarding an alignment alternative, which crosses an existing road with an overly acute angle but would be relatively good if an existing road is re-optimized and the costs for being re-optimized are small. The developed local intersection optimization can be used for improving search flexibility, thus allowing more effective intersections. It also provides a basis for extending alignment optimization from single highways to networks.

Throughout case studies, several optimized solutions are found based on different artificial and real case studies. Although there are enormous numbers of possible cases to be checked and each case needs a sufficiently large number of replications, the obtained solutions are found to be at least reasonable and satisfactory based on the locations and cost estimates for each cost item.

Computation time is a function of various variables: algorithms involved, types of study areas, number of generations, number of properties and initiation of local optimization of intersections. The longest computation time, approximately 19 hours and 7 minutes, is found in the last case study where a real GIS map is used and local intersection optimization is involved as well. From these results, it can be argued that the computational efficiency of the developed methods is acceptable as we consider the months or even years long engineering processes in real highway construction projects.

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