ABSTRACT

Title: AN ALIGNMENT OPTIMIZATION MODEL FOR A SIMPLE HIGHWAY NETWORK

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A new highway addition to an existing road network is typically considered for improving traffic performance in that road network. However, finding the new highway that best improves the existing network is a very complex problem since many factors affect the road construction. Besides changes in traffic flow patterns due to the new highway, various costs associated with highway construction as well as design specifications, safety, environmental, and political issues affect such a project.

Until recently, many studies have dealt separately with the problems of highway alignment optimization and network design. However, no models have been found that integrate these problems comprehensively and effectively. This dissertation seeks to find a realistic three-dimensional highway alignment that best improves an existing network, while considering its costs, geometric design, and environmental impacts on the study area. To fulfill this objective, an effective network model is developed that can simultaneously optimize (i) highway alignments and (ii) junction points with existing roads. In addition, the model's optimization process considers traffic impacts due to the highway addition as well as factors associated with its construction.

This dissertation starts by investigating the major cost components and important constraints in the highway design processes. Next, existing models for optimizing highway alignments are reviewed by assessing their advantages and disadvantages. Effective solution search methods are then developed to help solve the complex optimization problem. Development of the search methods is essential since an equilibrium traffic assignment as well as alignment optimization is undertaken in the proposed network model. Precise formulations of various highway costs and constraints are also developed for evaluating the various candidate alternatives. Cost functions for system improvements that can be obtained from the new highway addition are proposed. These are calculated based on the equilibrium traffic flows found from the assignment process. Complex geographical constraints including user preferences and environmentally sensitive areas are realistically represented, along with design standards required for highways. To represent highway alignments, sets of tangents, circular curves and transition spirals are used; in addition, three-leg structure models are also developed for representing the highway endpoints. Finally, several case studies are conducted to test the performance of the proposed models.

AN ALIGNMENT OPTIMIZATION MODEL FOR A SIMPLE HIGHWAY NETWORK

By

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DEDICATION

This dissertation is dedicated to my beloved wife and parents.

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Finally, I want to deeply acknowledge my family's endless love and support on me. No words can describe my appreciation to my beloved wife, Hyeyoung Han, who came to this foreign country to support me and sacrificed herself for her husband's study. I owe a great debt to her. I also want to express great appreciation to my respectable parents, who give me enormous love and prayer and always proud of me. I believe that they congratulate my achievement the most in the world.

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Chapter 1: Introduction

1.1 Background and Research Motivation

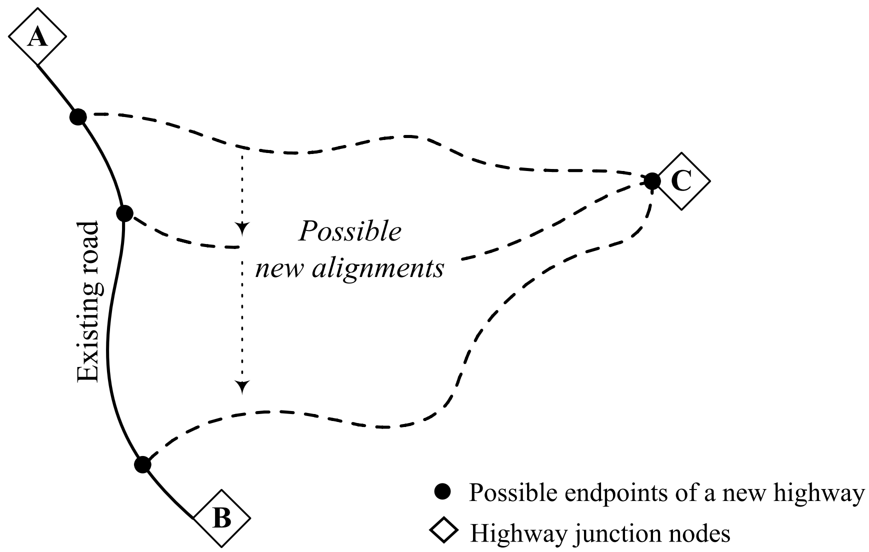
The addition of a new highway to an existing road network may be considered to improve the performance of that road network. The network users may thus save travel time, vehicle operating and accident costs, and the system operators (e.g., local governments) may obtain positive economic impacts. However, finding the new highway that best improves the existing road network is a very complex problem since many factors affect the road construction. Besides changes in network traffic flow patterns from the new highway addition, various costs associated with highway construction as well as design specifications, safety, environmental, and political issues affect such a project.

In the conventional highway design process, highway planners and engineers select only several candidate alignments, and then narrow their focus to the detailed alignment design. However, there may exist many possible alternatives that should be considered, as Figures 1.1 and 1.2 suggest. Furthermore, considerable time and cost is needed to find the best one among the candidate alignments since the conventional approach requires repetitive manual processes for performing detailed design and evaluating the all alignments.

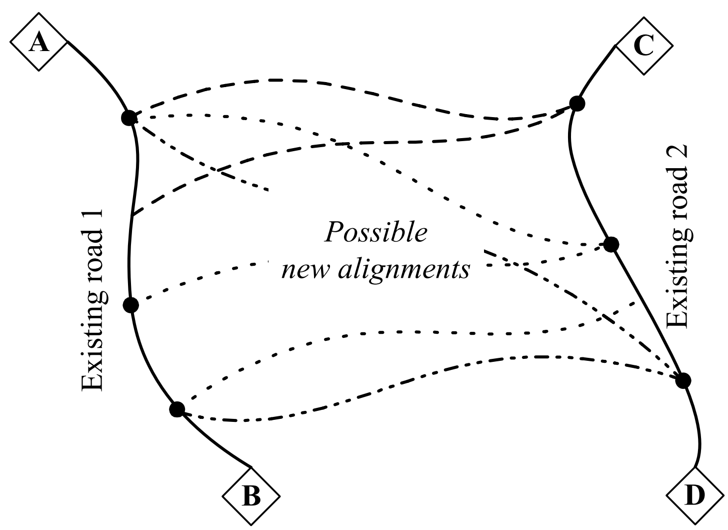
To overcome such complex and time-consuming limitations in the traditional highway design process, many studies have proposed automated and computerized highway-design models, such as *dynamic programming*, *network optimization*, and *genetic algorithms (GA's)*. (Chapter 2 provides detailed discussions of previously developed alignment-optimization models.) However, some of those models, although performing well in certain aspects, still have considerable weaknesses and are not widely utilized in real world applications. They may require special data formats, consider only limited number of factors associated with highway construction, or provide unrealistic highway alignments for real world applications with strong underlying assumptions; some models, despite providing relatively good solution alignments, may have significant computation burdens due to inefficient solution search methods.

Through this dissertation we try to optimize highway alignments added to an existing (simple) road network, by developing effective solution search methods coupled with precise formulation of various highway costs and constraints; examples of the simple road network considered in this study are presented in Figure 1.3. Note that one type of widely used metaheuristics, namely genetic algorithms (GAs) are employed to solve this complex and time-consuming problem.

It is expected that highway planners and designers may greatly benefit from the proposed network model, which offers well optimized candidate alternatives developed with automated GIS data extraction and comprehensive evaluation procedures rather than merely satisfactory alternatives in the planning stages of new highways. Problem definition and research objective and scope of the model are discussed in the following sections.



(a) Case 1: Either Start or End Point of a New Alignment Is Undefined



(b) Case 2: Both Endpoints of a New Alignment Are Undefined

Figure 1.1 Possible Highway Alignments Connecting Existing Roads

1.2 Problem Statement

Suppose that construction of a new highway is considered to improve the traffic performance of an existing highway network. Then, highway planners and designers will try to find an economical path that minimizes the total construction cost as well as improves as well as possible the traffic performance of the network, while satisfying geometric design, operational, and geographical constraints (including user preferences[1]). This dissertation seeks to find such an economical path.

It is noted that horizontal and vertical profiles of a new highway may significantly vary depending on the locations of its endpoints[2] (refer to Figures 1.1 and 1.2) as well as factors associated with its construction (such as topography and land-use of the study area and its design standards). In addition, changes in traffic flow patterns of the road network may also vary depending on where the alignments are connected on that network and their total distances. Until now, however, no models have been found that jointly consider these issues. This dissertation takes such considerations into account in the alignment optimization process. The basic simplifying assumptions of the proposed network model are described as follows:

Basic Assumptions

1. A new highway addition to an existing (simple and small) road network does not significantly affect overall system demands for a short-term analysis period (i.e., the given O/D trip matrix is assumed to be identical with and without a new

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[1] User preferences may include preferences of highway planners and designers or opinions from public hearing that affect right-of-way of a new highway.

[2] i.e., where to connect a new highway in the existing road network

highway); however, the new highway may affect the route choice of motorists (i.e., the network users can freely select their travel paths).

2. Two types of user classes (auto and truck) are used for evaluating the user costs of the road network.
3. There is no significant difference in economic development impacts from the various highway alternatives.
4. Traffic operates only through the analyzed road network.
5. A new highway is connected with existing roads and there are preferred road segments for its possible endpoints.
6. Design standards of the new highway are consistent along its alignment.

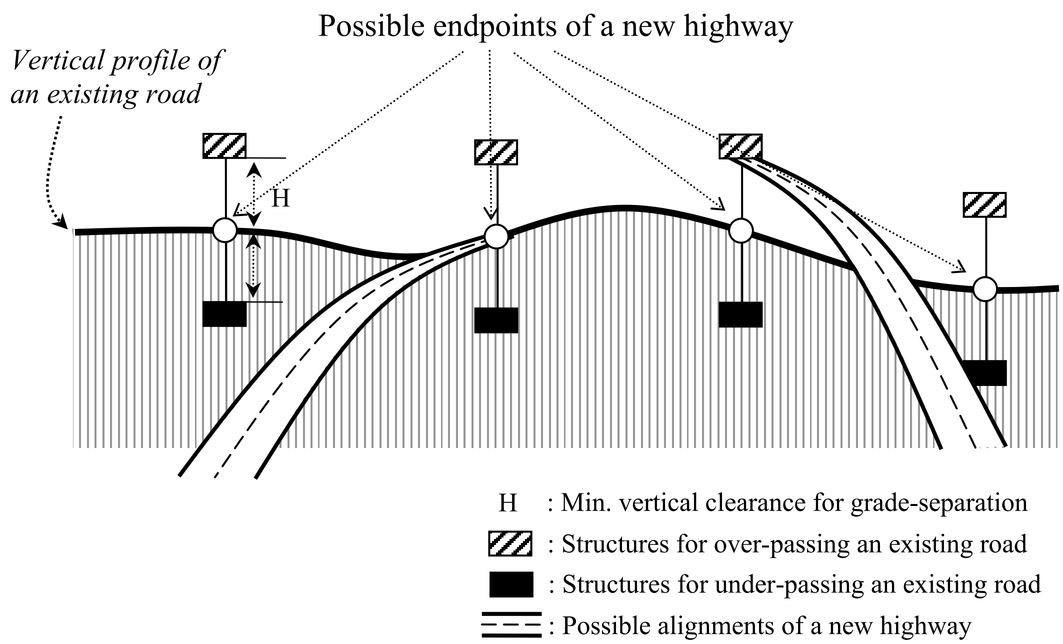

Figure 1.2 Possible Highway Endpoints along an Existing Road

1.3 Research Objective and Scope

The major objective of this study is to develop an effective optimization model that deals with the network problem defined in Section 1.2. To achieve such objective, this dissertation pursues several research goals listed as follows:

1. Develop a model framework for comprehensively optimizing highway alignments:
    A. Optimize highway junction points (including endpoints and intersection points) with existing roads besides alignment optimization.
    B. Formulate cost functions for evaluating system (network) improvements due to the new highway additions besides detailed environmental costs as well as construction costs.
    C. Integrate a traffic assignment process with the alignment optimization process to obtain equilibrium traffic flows of the network updated with the new highway additions.
2. Improve feasibility of generated solutions in order to enhance computational efficiency and solution quality of the alignment optimization process.
3. Realistically represent complex geographical constraints (including environmentally sensitive areas and user preferences) in addition to highway design constraints.
4. Represent three-dimensional (3D) highway alignments realistically.
5. Model three-leg structures for the highway endpoints (including their geometric designs and cost functions).

Note that this dissertation extends previous studies by Jong (1998), Jha (2000), Kim (2001), and Jha et al. (2006). Their work is adapted here to solve the more complex network optimization problem. A brief review and some limitations of their work which is relaxed in this dissertation are summarized in sections 2.2.2 and 2.2.3, respectively.

In order to solve the proposed network problem, a bi-level programming structure[3] is introduced in this study; (i) the highway alignment optimization (HAO) problem is considered as the upper-level problem and (ii) the traffic assignment problem is regarded as the lower-level problem of the model structure. The concept of this bi-level model structure is that (1) traffic impacts of new highways to the existing road network are based on traffic assignment results (the output of the lower-level problem) and (2) they are evaluated together with other highway costs (such as construction costs and environmental costs) during the optimization process (see section 6 for the detailed model structure).

Development of the objective function requires precise formulation of various alignment-sensitive costs (such as, right-of-way and earthwork costs of the new alignment) as well as system improvements that can be obtained from the highway

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[3] A problem where an optimization problem is constrained by another one is classified as a *bi-level programming problem* (Floudas et al., 1999). According to Yang and Bell (1998), many decision-making problems for transportation system planning and management can be described as a leader-follower game where the transportation planning departments are leaders and the users who can freely choose the path are the followers; normally it is assumed that transportation planning managers can influence, but cannot control the users' route choice behavior. Such an interaction is normally represented as a *bi-level programming problem* in many studies on those subjects.

additions (such as, reduction in travel time and vehicle operating cost of the network). In developing the model's constraints, complex geographical constraints, including user preferences and environmentally (or politically) sensitive areas, should be realistically represented and coupled with the design specifications required in highway construction. A good representation of such constraints may greatly reduce the alignment search problem by excluding many possibilities and requiring alignments to pass through some narrow "gates" or "corridors".

Improving the feasibility of solutions generated from evolutionary search algorithms (genetic algorithms (GAs) are used in the model) is also crucial for enhancing computation efficiency of the proposed model. Since the model has to deal with the traffic reassignment process iteratively (for different alternatives) besides the alignment generations and evaluations, computational efficiency is an important issue in the optimization process. Two effective constraint handling methods are developed in this dissertation for such purposes (see chapters 3 and 4).

To realistically represent highway alignments, incorporation of transition spirals coupled with circular curves is highly desirable in the horizontal curved sections of the resulting highways, and three-leg structures (e.g., three-leg intersections and trumpet interchanges) should also be modeled for representing the highway endpoints.

Figure 1.3 shows an example highway network considered in this dissertation, which may be encountered in real world situations. In the figure, a new highway alignment is added to the network, and its start and end points are placed along existing roads. The new alignment can intersect existing roads at multiple points so that several possible road segments (here three) constitute it. In the proposed model, as stated earlier, we allow the network users to travel to the new alignment by incorporating highway cross-structures at the intersection points with the existing roads. Note that locations of the intersection points (including the endpoints of the new highway) are iteratively updated whenever a new alignment is generated, and they are reflected in the traffic assignment process for finding equilibrium traffic flows of the updated network.

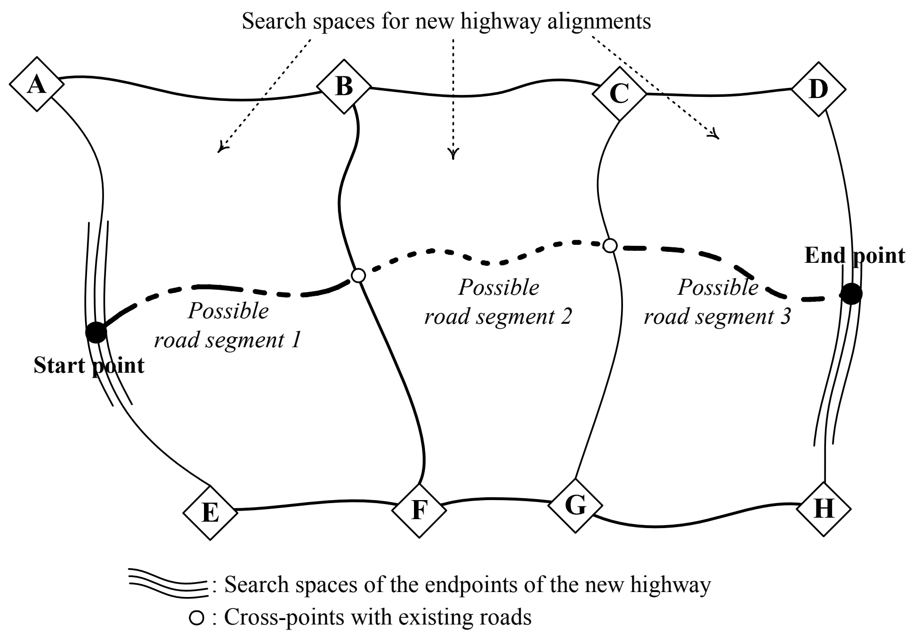

Figure 1.3 An Example Road Network with Addition of a New Highway Alignment Consisting of Possible Road Segments

A more complex example which may be encountered in real-world situation is presented in Figure 1.4. Incorporation of a high-performance computing technique (such as parallel computing) may be necessary to deal with such a complex case or an even more complex network.

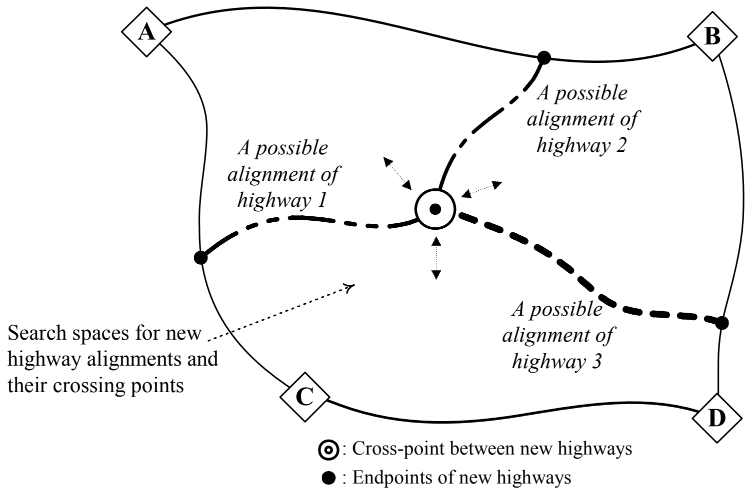

Figure 1.4 A Possible Simple Network Connecting Existing Roads with Three Endpoints and One Junction Points between New Alignments

1.4 Research Approach

The research approach is quite straightforward. A series of steps specified below shows how the proposed optimization problem can be solved. These steps (also shown in Figure 1.5) basically outline the framework of this study.

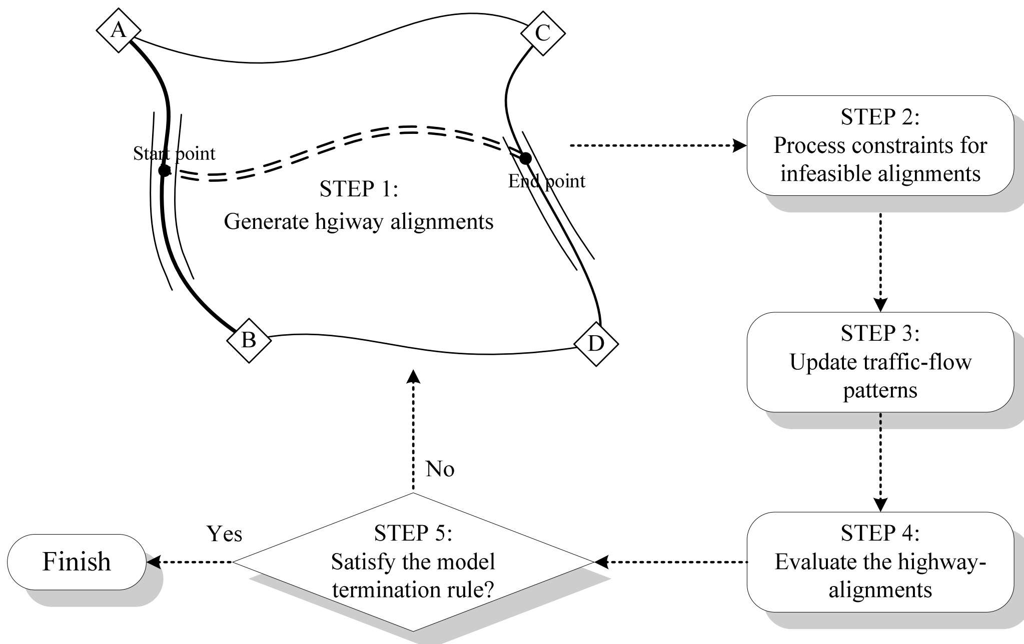

Figure 1.5 Concept of Alignment Optimization Process for a Simple Highway Network

STEP 1: Generate a highway alignment connecting existing roads.

- The endpoints of the new highway are selected along specified existing road segments or selected from a discrete set of points.
- A three-dimensional (3D) highway alignment connecting the endpoints is created both horizontally and vertically.

STEP 2: Process constraints determining infeasible solution alignments.

- The alignments violating model constraints are controlled by the constraint handling methods developed in this dissertation.

STEP 3: Update network configuration and equilibrium traffic flow patterns.

- Given information of the existing road network (such as, travel demands (O/D trip matrix) and link characteristics) and of the new highway alignment added (start and end points and crossing points with existing roads), find the equilibrium traffic flows using the traffic assignment procedure.

STEP 4: Evaluate costs associated with the candidate alignment.

- Compute the highway construction cost.
- Compute the network user cost saving (comparison before and after the highway addition).

STEP 5: Check the model termination rule.

- If the model termination rule is satisfied, finish program.
- Otherwise, go to STEP 1.

To perform the above five steps, thorough studies are needed. For step 1, (i) an endpoint determination procedure and (ii) an alignment generation procedure are proposed. In addition, efficient alignment search methods based on Jong's (1998) customized genetic algorithms (GAs) are developed. The search methods are used to realistically represent complex user preferences (e.g., outside environmentally sensitive areas) in the geographical search space as well as to enhance model computation efficiency, by avoiding unnecessary computation time for evaluating infeasible solution alignments.

For step 2, good constraint handling techniques (combination of direct and indirect methods) are needed to efficiently guide the search process. Such methods are used to handle generated alignments that violate design and geographical constraints.

For step 3, as stated earlier, a traffic assignment process is incorporated in the model. The equilibrium traffic flows updated every iteration are used for estimating cost of the network users.

For evaluating the candidate alignment in the step 4, the model objective function could be formulated for either cost minimization or net benefit maximization. In this study, in which the overall origin-destination flows are assumed to remain unaffected by new alignments, cost minimization is sufficient. All important alignment-sensitive costs and user costs are included in the objective function.

1.5 Organization of the Dissertation

The organization of this dissertation is as follows:

Chapter 1 introduces the research background and motivation, the problem definition, the research objectives, and the research approach. Chapter 2 reviews the literature on (i) various cost components and constraints associated with highway construction, previous models developed for (ii) the highway alignment optimization problem as well as (iii) the discrete network design problems, and (iv) various constraint handling techniques used for evolutionary algorithms (particularly for genetic algorithms).

The remaining chapters of this dissertation are grouped into three parts (I, II, and III). Part I (Chapters 3 and 4), discusses computational efficiency and solution quality issues in the highway alignment optimization (HAO) problem, which is the upper-level problem of the proposed network optimization problem. Feasible Gate (FG) approaches are proposed in Chapter 3, and Prescreening and Repairing (P&R) methods are described in Chapter 4.

In Part II (Chapters 5 to 7), tasks required for modeling the network optimization problem are discussed. Highway alignments and endpoints are realistically represented in Chapter 5. The basic model structure (the bi-level programming structure) of the network problem and its optimization procedure are discussed in Chapter 6. Various highway cost items constituting the objective function of the problem are formulated in Chapter 7.

Part III (Chapters 8 and 9) presents case studies and summary of this dissertation work. Model applications to real highway projects are described in Chapter 8 (e.g., application procedures, results, and discussion), while model capabilities and research contributions are summarized in Chapter 9

Chapter 2: Literature Review

The literature review for this study includes four sections. Cost items and constraints that are normally considered in highway construction are described in the first section. Models for optimizing highway alignments are reviewed in the next section with particular attention to the highway alignment optimization (HAO) model, which is the predecessor of the proposed model. In the third section, various constraint handling techniques used in evolutionary algorithms (particularly for genetic algorithms) are investigated. Models for the discrete network design problem, which is another major research area associated with highway improvement problems, are briefly reviewed in the fourth section. A summary of findings from the literature review is provided in each section of this chapter.

2.1 Costs and Constraints Associated with Highway Construction

This section investigates major cost items and constraints that should be considered in highway construction. Such an investigation is essential since these are the criteria that are most commonly considered in evaluating highway alternatives.

2.1.1 Highway Costs

Many cost components directly or indirectly affect in construction of new highways. Besides the initial construction costs, which are directly related to highway construction (e.g., earthwork, land acquisition, pavement and drainage), user costs and environmental costs should also be considered for highway construction projects.

According to Jha (2000), it is important that all dominating and alignment sensitive costs should be considered and precisely formulated for a good highway optimization model; dominating costs are those which make up significant fractions of the total cost of a new highway alignment, and alignment sensitive costs are those which vary with relatively slight changes in alignment geometries. Normally, highway user costs (such as travel time cost and vehicle operating cost) are the most dominating ones as they persist over the entire design life time of the highway and the users' value of time is usually higher than other costs associated with highway construction. Structure costs (e.g., bridges and interchanges construction costs) and earthwork costs may dominate if a highway is constructed in a mountainous area. A highway passing through an urban area may have a high percentage of right-of-way cost, since the required land acquisition cost of that area may be relatively higher than other costs.

A number of studies (Winfrey, 1968; Moavenzadeh et al., 1973; OECD, 1973; Wright, 1996; Jong 1998; Jha 2000; Kim, 2001) have discussed highway costs. Five main categories of the major highway transportation costs were itemized by Winfrey (1968), OECD (1973), and Wright (1996) as shown in Table 2.1. Jong (1998) also discussed the cost items shown in Table 2.1 in his dissertation. He formulated mathematical highway cost models based on the classifications, and incorporated them into his alignment optimization model; the highway planning and design costs are not considered in this model.

Table 2.1 Classification of Highway Transportation Costs

Classification		Examples
Construction Costs		Earthwork, Pavement, Right-of-way
Operation and Management Costs		Pavement Mowing, Lighting
User Costs	Vehicle Operating Cost	Fuel, Tire wear, Depreciation of vehicles
	Travel Time Costs	Vehicle hours times unit value of time
	Accident Costs	Predicted number of accidents times accident unit cost
Environmental Costs		Noise, Air pollution, Wetland loss
Planning and Design Costs		Consulting and Data collection

Source (Jong, 1998)

Comprehensive highway cost models were also proposed by Moavenzadeh et al. (1973). A life-cycle concept was introduced in the model formulation of highway construction, vehicle operating, and maintenance costs. Readers may refer to Jha (2000) and Kim (2001) for input requirements and discussion of the model developed by Moavenzadeh et al. (1973).

Construction Costs

The construction costs are the major agency costs that directly affect local government authorities or highway consultant companies. According to the Maryland State Highway Administration (MSHA, 1999), the road construction costs can account for up to 75 % of the total highway cost. Normally, costs required for earthwork, pavement, right-of-way, structures (e.g., bridges and interchanges), and miscellaneous items (such as fencing and guardrails) are included in this category. Jong (1998), Jha (2000), and Kim (2001) reclassified them into the following four sub-categories based on the characteristics of each cost component:

a. Volume-Dependent Cost
b. Location-Dependent Cost
c. Length-Dependent Cost
d. Structure Cost

Such a classification is quite useful for quantifying the construction costs and representing them in the alignment optimization process. The earthwork cost belongs to the volume-dependent cost since it can be quantifiable based on the amount of earthwork volume required for highway construction, and some unit costs related to the earthwork (such as unit embankment and excavation costs) may be needed to estimate the cost. The right-of-way cost, including land acquisition costs and property damage and compensation costs, are included in the location-dependent costs (Jha, 2000). The length-dependent cost is defined as the cost proportional to alignment length. Pavement cost and road superstructure and substructure costs (such as fencing, guardrails, and drainage costs) can be included in this category. In highway engineering, structures normally include bridge, tunnel, interchange, intersection, and over or underpasses. Costs required for building those structures belong to the structure cost category.

Note that all these costs are dominating and alignment-sensitive costs that should be included in the optimization process. Readers may refer to Moavenzadeh (1973), Jong (1998), and Jha (2001) for more detailed discussion of the construction costs.

Highway Operation and Maintenance Costs

The highway operation and maintenance costs occur throughout the life of the road alignment. Therefore, these costs are generally discounted over the alignment life at a certain interest rate for estimating them at the initial stage of road construction (Jong, 1998). These costs may include preventive maintenance costs (such as costs required for repairing roadway pavement, guardrail, and median) and even road rehabilitation costs.

User Costs

The highway user costs are sometimes also called traffic costs and usually include travel time, vehicle operating, and accident costs. In a highway improvement project, these costs are normally used for a user benefit analysis, by comparing their values estimated before and after the project. The travel time cost can be computed with the users' travel time estimated in a certain condition (e.g., specific time and scenario (with or without a new highway)) of a highway network and their value of time estimated externally. The vehicle operating costs typically include estimated fuel consumption and vehicle depreciation costs. The accident costs are usually estimated with unit accident cost and accident rates predicted from an accident regression analysis.

Note that the user cost items are the dominating costs, and they are sensitive to alignment length as well as to the locations where a new alignment is connected to existing road networks. Therefore, the user cost should also be considered in the optimization process. The methods for estimating these costs are well discussed in the AASHTO manual for "User Benefit Analysis for Highways" (2003).

Environmental Costs

Construction of a new highway may also significantly affect environmentally sensitive areas (such as wetlands and historic areas) and human activities of the existing land-use system, and even may cause air pollution and increased noise level. The environmental impacts of the new highway construction are often considered as the most important issues in the modern highway construction projects; hence, these costs should also be accounted for in the alignment optimization process.

Jong et al. (2000) consider the environmental impacts of highway alternatives in the alignment optimization problem by using a penalty concept; they assign high penalties to the areas considered as the environmentally sensitive regions. However, it should be noted that a detailed trade-off analysis (or a decision making process) may be required to use the penalty concept if the project area is very complex so that there are different levels of importance in the environmentally sensitive regions. An example of the trade-off analysis applied in a real highway construction project may be found in Kang et al. (2005). Jha (2000) provides more detailed discussions for the environmental issues associated with the new highway construction. He comprehensively formulates highway environmental costs in the alignment optimization process with a GIS-based application.

Kim (2001) considers the noise and air-pollution effects of the new highway alternatives in the optimization process although they are not significantly sensitive to highway alignments. A previously developed noise model (Haling and Cohen, 1996) and air-pollution cost model (Halvorsen and Ruby, 1981) are adopted to represent the noise and air-pollution effects on solution selection.

Planning and Design Costs

The planning and design costs may be neglected in alignment optimization problems because they are insensitive to various highway alternatives. Furthermore, their impacts on the total cost of a highway construction project are not significant (i.e., not dominating).

2.1.2 Constraints in Highway Construction

Normally two types of constraints are considered in new highway construction. These are (i) design constraints and (ii) environmental and geographical constraints. The former constraints are usually based on AASHTO design standards (2001); however, the latter ones are sensitive to many complex factors associated with topology, land-use of the project area, and even preferences of decision makers.

Design Constraints

Basically, the geometric design of a highway determines the horizontal alignment, vertical alignment, and cross-section of that highway. The horizontal alignment of a highway, which is the projection of a three-dimensional (3D) highway onto a two-dimensional horizontal surface (i.e., XY surface), generally consists of three types of design elements: tangent segments, circular, and transition curves. The vertical alignment of a highway is the projection of a design line on a vertical plane as if all horizontal curves were stretched to straight, and composed of a series of grades joined to each other by parabolic curves. The most important design constraints (from

AASHTO, 2001) required for constructing the horizontal and vertical alignments are as follows:

1. Minimum horizontal curve radius
2. Sight distance on a horizontal curve
3. Minimum superelevation runoff lengths (only if transition curves are considered as a part of the horizontal curved section)
4. Maximum gradient
5. Sight distance on crest and sag vertical curves (i.e., minimum length of crest and sag vertical curves)
6. Minimum vertical clearance for highway crossing and bridge construction

Geographical and Environmental Constraints

Besides the design constraints stated above, geographical and environmental constraints should also be considered in the highway design process. These constraints are often regarded as the most important issues in real highway construction projects, and vary with the different communities affected by projects. These are categorized as follows:

7. Environmentally sensitive areas; i.e., no-go areas (e.g., wetlands and historic districts)
8. The areas outside interest (i.e., control areas defined by highway designers and planners)
9. Fixed point (or area) constraints through which the alignment must pass

2.2 Alignment Optimization Problem

Alignment optimization is a complex non-linear optimization problem whose objective functions and constraints are normally noisy and non-differentiable. Many important costs and complex constraints associated with road construction project are normally included in the problem.

Table 2.2 Studies on Highway Alignment Optimization

Target for optimizing	Type of approach	References
Horizontal alignment only	Calculus of variations	Howard et al. (1968), Shaw and Howard (1981 and 1982), Thomson and Sykes (1988), and Wan (1995)
	Network optimization	Turner and Miles (1971), OECD (1973), Athanassoulis and Calogero (1973), Parker (1977), and Trietsch (1987a and 1987b)
	Dynamic programming	Hogan (1973) and Nicholson et al. (1976)
	Genetic algorithms	Jong (1998) and Jong et. al (2000)
Vertical alignment only	Enumeration	Easa (1988)
	Dynamic programming	Puy Harte (1973), Murchland (1973), Goh et al. (1988), and Fwa (1989)
	Linear programming	Chapra and Canale (1988) and Reville et al. (1997)
	Numerical search	Hayman (1970) and Robinson (1973)
	Genetic algorithms	Jong (1998), Fwa et al. (2002), and Jong and Schonfeld (2003)
Three-dimensional (3D) alignment	Numerical search	Chew et al. (1989)
	Network optimization	de Smith (2006)
	Neighborhood search heuristic with MIP	Cheng and Lee (2006)
	Genetic algorithms	Jong (1998), Jha (2000), Kim (2001), Jong and Schonfeld (2003), Chan and Fan (2003), Jha and Schonfeld (2004), and Jha et al. (2006)

Sources: adapted from Jong (1998) and Kim (2001)

Its objective is to find the best alignment (usually the most economical path) for a new highway connecting given endpoints, and there are an infinite number of possible alignments to be evaluated. Many researches have been conducted on the subject, and they are listed in Table 2.2.

2.2.1 Solution Search Methods Used in Alignment Optimization

Several solution search methods (at least seven so far) have been developed either for optimizing horizontal alignments or vertical alignments or both for three-dimensional (3D) highway alignments (see Table 2.2). These are *calculus of variations*, *network optimization*, *dynamic programming*, *enumeration*, *linear programming*, *neighborhood search heuristic with mixed integer programming*, and *genetic algorithms*.

Jong (1998), Jha (2000), and Kim (2001) comprehensively reviewed such optimization approaches that had been proposed by early 2000. According to their assessments, all the methods other than genetic algorithms (GAs) have significant defects for the alignment optimization problem of which cost functions are non-differentiable, noisy, discontinuous, and implicit. Those defects are summarized in Table 2.3.

GAs are adaptive search methods based on the principles of natural evolution and survival of the fittest. They can avoid getting trapped in local optima and can find very good solutions in a continuous search space, while providing pool-based search rather than single solution comparison as in other heuristics (e.g. simulated annealing and Tabu search). The effectiveness of approaches based on GAs for the alignment optimization problem can be described in terms of the following key advantages:

a. Can yield realistic alignments
b. Allows a continuous search space
c. Can find globally or near globally optimal solutions
d. Can consider most of the important constraints and costs
e. Can simultaneously optimize horizontal and vertical aignments
f. Can handle alignments with backward bends (Jong, 1998)

Recently, since the GAs were first used in the highway alignment optimization problem by Jong (1998), such a meta-heuristic method has been widely applied to such problems (Jong, 1998; Jha, 2000; Kim, 2001, Fwa et al., 2002; Jong and Schonfeld, 2003; Chan and Tao, 2003; Jha and Schonfeld, 2004; Jha et al., 2006); only one method other than the GAs (i.e., a neighborhood search-heuristic with mixed integer programming (MIP) by Cheng and Lee (2006)) has been found on the subject.

As in the GA-based approach, Cheng and Lee's method can also provide many benefits in finding the solution alignments. For instance, it can (i) yield a realistic alignment, (ii) can consider many important design constraints, and (iii) can allow continuous search space as well. However, since the method is used for two-stage optimization (find horizontal alignment, and then optimize vertical alignment), it is hard to obtain global optima. A more detailed review of Cheng and Lee's (2006) model together with the other models for optimizing three-dimensional highway alignments is presented in the next section.

Table 2.3 Defects of Search Methods Used in Existing Alignment Optimization Models

Method	Defects
Calculus of Variations	-Cannot deal with discontinuous cost items -Requires complex modeling and heavy computation efforts -Has tendency to get trapped in local optima
Network Optimization	-Cannot yield smooth alignments -Uses discrete solution set rather than continuous search space -Requires large memory
Dynamic Programming	-Cannot yield smooth alignments -Uses discrete solution set rather than continuous search space -Requires large memory
Enumeration	-Is inefficient -Uses discrete solution set rather than continuous search space
Linear Programming	-Cannot yield smooth alignments -Must have linear form for all cost items
Numerical Search	-Has tendency to get trapped in local optima -Requires complex modeling and heavy computation efforts -Has difficulty in modeling discontinuous cost items
Neighborhood search-heuristic with MIP	-May produce local optima from the conditional optimization

Source: adopted from Jong (1998)

2.2.2 Models for Optimizing Highway Alignments

Three types of model for optimizing highway alignments are found in the literature (see Table 2.2). These are (i) horizontal alignment optimization models, (ii) vertical alignment optimization models, and (iii) models for optimizing three-dimensional (3D) alignments (i.e., optimizing both horizontal and vertical alignments). As shown in the table, much progress has been made in developing models for optimizing vertical alignments over the past three decades. The progress in developing models for optimizing horizontal alignments or 3-dimensional alignments is very limited and the number of such models is small. The main reason is that modeling horizontal alignments is quite complex and requires substantial data for various cost components, such as right-of-way cost (i.e., land-acquisition cost) and pavement cost, and other political or environmental issues. Furthermore, the horizontal and vertical alignments are interrelated in complex ways, in design elements as well as costs associated with construction.

Models for Optimizing Three-Dimensional Highway Alignments

In the literature, only four distinct models for optimizing 3D highway alignments are found (see Table 2.2). These are (i) a model developed by Chew, Goh, and Fwa (1989), (ii) the highway alignment optimization (HAO) model by a research team from University of Maryland (Jong, 1998; Jong and Schonfeld, 2003; Jha and Schonfeld, 2004; Kim et al, 2004; Jha et al., 2006), (iii) a 3D model proposed by de Smith (2006), and (iv) that developed by Cheng and Lee (2006).

A. Model of Chew, Goh, and Fwa (1989)

Chew, Goh, and Fwa (1989) proposed the 3D highway alignment model as the extension of their continuous model for optimizing vertical alignment (Goh, Chew, and Fwa, 1988). The authors described the highway alignment with a 3D cubic spline polynomial curve, and used the quasi-Newton decent algorithm (one of the numerical search methods) to search the solution alignment. However, the alignment resulting from the model is far from realistic since no highway design element is considered in the alignment. Furthermore, the solution search algorithm embedded in the model only guarantees local optimum. Another potential defect of the model is that it seems difficult to add discontinuous functional forms into the objective function because the solution algorithm requires a differentiable objective function (Jong, 1998); note that usually the functional form of alignment's right-of-way and environmental costs are not continuously differentiable (i.e., non-exact).

B. The HAO Model[4] (from 1998 to present)

The highway alignment optimization (HAO) model has been extensively refined in recent years to find the 3D highway alignments that best satisfy various objectives and constraints for use in the initial stages of road construction projects. It should be noted that the model is the only one that can simultaneously (i.e., jointly) optimize horizontal and vertical alignments.

In the model, genetic algorithms (GAs) with a number of specialized genetic operators are used for optimizing alignments (Jong and Schonfeld, 2003), and a Geographic Information System (GIS) is integrated with the GAs to realistically reflect a real world problem (Jha and Schonfeld 2000). While the model runs, the GAs and GIS communicate (bi-directionally) by transmitting the model inputs and outputs. In the model, the GIS is primarily used for right-of-way (ROW) cost calculation and environmental impact assessments of the solution alignments, while the GAs are used for:

a. Generation of points of intersection (PI's) for horizontal and vertical alignments simultaneously;

[4] Jong (1998), Jha (2000), and Jong and Schonfeld (2003) show the effectiveness of the GA-GIS based search in alignment optimization problems with several case studies coupled with various sensitivity analyses; they show the capabilities of the pool-based search of the GAs, by finding near optimal solutions in a continuous search space (2D and even 3D) without getting trapped in local optima.

b. Optimal search based on the principles of natural evolution and survival of fittest

A chronological sequence of the model's development is given below:

Jong (1998) proposed genetic algorithms (GAs) as a solution search method for optimizing the highway alignments. He represented the alignments in 3D continuous space, while considering major highway design elements (tangents and circular curves for horizontal alignments; gradients and parabolic curves for vertical alignments); note that no transition curves were considered for representing horizontal alignments. Various highway cost models for alignment sensitive costs (such as earthwork, right-of-way, and length-dependent costs) were also developed for use them in the model objective function. For handling the infeasible solutions violating model constraints, a static penalty function was employed in the model.

Jha (2000) as well as Jha and Schonfeld (2004) extended Jong's model by integrating a GIS analysis. This allows the model to more realistically estimate right-of-way costs and consider environmental impacts of the resulting alignments. The model can be applied to real highway project directly with the integration of the GIS; note that despite the great usefulness of the GIS, no other studies have been found that use the GIS in the context of the alignment optimization problem. Kim (2001) as well as Kim and et al. (2004) further extended the model by incorporating some major structure costs in the model objective function.

Recently, the HAO model has been applied to an actual road construction project, the Brookeville Bypass (Kang et al., 2006), in which the computational results are compared against those obtained manually, while providing practical information (e.g., station coordinates of optimized horizontal and vertical alignments and total construction cost) to highway engineers and planners. Through the real world application of the model, it has been recognized that the model can still benefit from various improvements (e.g., in computational efficiency and solution quality issues) despite its capability demonstrated in the Brookeville Bypass project.

Note that the network optimization model proposed in this dissertation is an extension of the HAO model, and is designed for dealing with the alignment optimization problem for a simple highway network as described in Section 1.2. Detailed discussions of the model limitations and strong underlying assumptions that should be relaxed for the extension are presented in Section 2.2.3.

C. Model of de Smith (2006)

The third distinct model for optimizing 3D alignments is that developed by de Smith (2006). The author proposed a gradient and curvature constraint method to determine an optimal alignment for roads, railroads, and pipelines. The model determines the optimal path based on four steps: (i) determination of initial shortest alignments that satisfy gradient constraints in a tilted planner surface, (ii) distance calculation of the alignments with elevation matrix, (iii) horizontal, and (iv) vertical path smoothing of the alignments with spline functions and curvature constraints. Detailed procedures are provided for all four steps. However, since his method may require a conventional cost evaluation procedure for candidate alignments from the first four stages and those steps may not be automatically integrated, considerable time may be needed to obtain the final solutions. de Smith also discusses ways of dealing with obstacles or no-go areas of the alignments and involves them as additional constraints in step 1. However, these are very rough and all bounds are parallel to the straight line between the start and endpoints of the alignment; thus, they cannot realistically represent real shapes of the untouchable areas on a nonplanar surface, as in a realistic GIS.

D. Model by Cheng and Lee (2006)

Cheng and Lee (2006) also proposed 3D alignment optimization model with a neighborhood search-heuristic for finding horizontal alignments and a mixed integer programming (MIP) method for finding vertical alignments. Several cost components (such as earthwork costs and bridge and tunnel costs) are included in the model objective function. The key contribution of this paper is that transition curves are used to realistically represent the curved sections of horizontal alignments while considering various design constraints associated with the curves. Besides the alignments profile (horizontal and vertical), a speed profile for heavy vehicles operating on the resulting alignments is created as a model output.

Despite the contribution, several limitations are found in the model. First (i) Cheng and Lee's (2006) model finds 3D highway alignments with a two-stage (conditional) approach; it sub-optimizes a horizontal alignment first, and then sub-optimizes the vertical alignment based on the horizontal alignment created. However, it should be noted that optimizing horizontal and vertical alignments (i.e., 3D alignments) simultaneously is clearly preferable to sub-optimizing the vertical alignment for a previously sub-optimized horizontal alignment since such a conditional optimizing process is less likely to avoid local optima. In addition, (ii) alignment right-of-way cost (which includes land acquisition and property damage costs), which is clearly dominating and sensitive cost in highway construction project, is not considered in the model. Note that it would be desirable that a model for optimizing highway alignment should directly exploit a GIS database because most spatial information is becoming available in such computer-readable form. This includes realistic shapes of land parcels, property values, and even various land-use patterns. Finally, in Cheng and Lee's (2006) model (iii) the untouchable area (i.e. no-go area of alignments) was assumed to be circular in shape although in reality it could have any shape.

Beyond the four distinct 3D models presented above, Chan and Tao (2003) also discussed the 3D alignment optimization model with the GAs and GIS-based approach. However, the methodologies used in their model, including alignment representation method with GAs and model formulation, are almost identical with those in the HAO model by Jong (1998) and Jha (2000). No significant difference, however minor, is found.

Models for Optimizing Vertical Alignments

As stated previously, many models for optimizing vertical alignments are found in the literature; the common optimization approaches used were *enumeration, dynamic programming, linear programming, numerical search,* and very recently *genetic algorithms (GAs)*. Here we introduce the most recent model on the subject by Fwa et al. (2002). Readers may refer to Tables 2.2 and 2.3 for review of the other models, or refer to Jong (1998) and Jha (2000) for more detailed discussion.

Fwa et al. (2002) optimize vertical alignments with the assumption that horizontal alignments are initially given (i.e., their model also process conditional optimization). They consider various design constraints on vertical alignments, such as gradient, curvature, fixed point, critical length of grade, and non-overlapping of horizontal and vertical curves; no constraints for horizontal alignments are considered. In the model, genetic algorithms (GAs) are used for optimizing vertical alignments, and a constant static penalty function is employed to handle the infeasible solutions generated from the GAs. A huge constant value (which is pre-specified as a model input) is added to the model objective function whenever the solution violates design constraints regardless of severity of the violation. It should be noted, however, that such a constant penalty function is generally inferior to a soft penalty function which adds more severe penalty with distance from the feasibility condition (Goldberg 1989; Richardson et al. 1989; Smith and Coit 1997). A huge constant value may cause serious errors by leading to large unsmooth steps during the optimization process, and thus may often fail to obtain optimal solutions. The penalty should be kept as low as possible for a smooth solution search process (Smith and Tate, 1993; Riche et al., 1995).

Models for Optimizing Horizontal Alignments

The common optimization approaches used on the subject were *calculus of variations, network optimization, dynamic programming*, and *GAs* (See Table 2.2). The progress in developing modes for optimizing horizontal alignment is slow, and no recent research has been found on the subject. Readers may refer to Tables 2.2 and 2.3 for a brief review of the previously developed horizontal alignment optimization models, or refer to Jong (1998) and Jha (2000) for more detailed discussion. Note that this dissertation seeks to optimize 3D highway alignments, and thus the investigation of the literature has focused on 3D highway alignment models rather than models for optimizing either horizontal or vertical alignments.

2.2.3 Summary

Through Section 2.2, many available alignment optimization models are reviewed. Among them, the HAO model (which have been extensively developed by Jong 1998; Jha 2000; Kim 2001; Jong and Schonfeld 2003) is the most attractive one that can possibly find globally or near globally optimal solutions (3D alignments) with customized GAs, while considering various geographical features with an integrated GIS application. As stated previously, this dissertation further extends the model to cover the network optimization problem by relaxing several assumptions in the model as well as developing efficient solution search methods. Further developments required for the model extensions are discussed below.

Further Development Required

Despite its many capabilities, several limitations of the HAO model are found from in reviewing it. These limitations include the following:

1. The model cannot evaluate the system improvement (e.g., travel time savings) that can be obtained from the new highway addition to the existing road network, outside the single alignment it optimizes.
2. The model does not consider changes in network flow patterns from the addition of different candidate alignments in the optimization process (i.e., traffic flows on the resulting alignments are given and fixed).
3. The start and end points of the alignments are given and fixed.
4. There are no transition curves in the horizontal curved sections of the alignments.

5. Structures for the endpoints of the alignments are not modeled.
6. The model is not computationally efficient enough since only penalty methods are used for handling many infeasible solutions generated from pure GAs.

The model estimates user costs based on traffic flows operating only on the alignment generated, regardless of those on other existing roads of which traffic flow patterns may be affected by the new highway addition; i.e., it does not consider changes in traffic flow patterns of the network from the new highway construction. However, since drivers' route choice patterns may be significantly affected by location and length of the new alignment, such assumptions should be relaxed. In order to do this, a traffic assignment process including many inputs required for the process (e.g., network information and origin and destination (O/D) trip matrix) should be added to the model. Furthermore, reformulation of the user cost functions is correspondingly required.

Another model limitation is that the start and end points of a new highway are assumed to be given and fixed before the optimization process. The fixed endpoint assumption can be relaxed to undefined endpoints by allowing the model to optimize the locations of the highway endpoints besides its alignment optimization. Such a relaxation is reasonable since the location of connections to an existing road network (i.e., the endpoints of a new highway) is not often determined in early stages of road construction projects. More importantly, the fixed endpoint assumption should be relaxed since traffic flow pattern on the network might be significantly changed depending on the locations of the endpoints.

There is no transition curve in the horizontal curved sections of the new alignment generated. However, incorporation of transition curves in the curved section is recommended to represent highway alignments more realistically. For designing high-speed highways in particular, the transition curves are strongly recommended.

Three-leg structures for the endpoints of the new highway (including their geometric designs and cost functions) should also be modeled to realistically represent its alignment in the optimization process. Trumpet interchanges, three-leg intersections and roundabouts, which are most widely used in the highway engineering, can be modeled.

Additionally, the model may be computationally expensive when dealing with problems requiring a complex and time-consuming evaluation process (e.g., the network problem proposed in this dissertation). The model uses only penalty methods for handling infeasible solutions. However, such a constraint handling method, despite its many advantages, wastes computation time since it has to evaluate all solutions including many infeasible solutions generated from the model. Various types of infeasible solutions (e.g., alignments violating design, geographical, and user-defined constraints) may be generated during the alignment optimization process. Detailed discussions of the constraint handling methods used in GAs are presented in the next section.

2.3 Constraint Handling Techniques for Evolutionary Algorithms

Evolutionary algorithms (EAs)[5], which constitute a subfield of artificial intelligence (AI)[6], have been developed for solving many complex constraint optimization problems or constraint satisfaction problems (CSPs)[7]. Normally, genetic algorithms (GAs), evolution strategies (ESs), evolutionary programming (EP), genetic programming (GP), and learning classifier system (LCS) are included in EAs. Among them, GAs are the most popular one that have many successful applications to complex problems (e.g., project scheduling problem and alignment optimization problem). It is noted that a key issue in the application of EAs (particularly for GAs) to a complex optimization problem is how to effectively handle the infeasible solutions (individuals) from the algorithms for a good solution search process. Such an issue is not simple since solution search techniques involved in GAs (such as reproduction, mutation, and recombination) are usually 'blind' to constraints. In other words, not all solutions from GAs are feasible. It is possible to generate a solution which does not satisfy the requirements of the problem (Michalewicz and Michalewicz, 1995).

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[5] "EAs are a class of direct, probabilistic search and optimization algorithms gleaned from the model of organic evolution." (Back, 1996)

[6] According to Rich (1983), artificial intelligence (AI) is the study of how to make computers do things at which, at the moment, people are better.

[7] "Constraint satisfaction problems (CSPs) are mathematical problems where one must find states or objects that satisfy a number of constraints or criteria. A constraint optimization problem can be defined as a regular CSP in which constraints are weighted and the goal is to find a solution maximizing the weight of satisfied constraints." (http://en.wikipedia.org/wiki)

Table 2.4 Typical Constraint Handling Methods Used in Evolutionary Algorithms

Control strategy	Approaches
Direct constraint handling	- Eliminating infeasible solutions - Repairing infeasible solutions - Preserving feasibility by special operators - Decoding (i.e., transforming the search space) - Locating the boundary of the feasible region
Indirect constraint handling	- Assigning penalty to objective of infeasible solutions

Source: Craenen et al. (2003) and Coello (2002)

Many constraint handling techniques for treating infeasible solutions of EAs have been developed. They are normally classified into two major categories in literature of computer science. Craenen et al. (2000) classified the constraint handling methods into two cases (direct and indirect constraint handling), depending on whether they are handled indirectly or directly. According to the authors, the "direct constraint handling" means that violating constraints is not reflected in the optimization objectives (i.e., fitness or objective function) so that there is no bias towards solutions satisfying them. On the contrary, the objective function includes penalties for constraint violation in case of the "indirect constraint handing" approaches. Typical approaches categorized in the two cases are summarized in Table 2.4, and their general advantages and disadvantages are presented in Table 2.5. Additionally, Table 2.6 shows several studies associated with each approach.

Table 2.5 General Advantages and Disadvantages of Direct and Indirect Constraint Handling Methods

Approach	Advantages	Disadvantages
Direct-constraint handling	- Might perform very well with significant improvement on computation efficiency - Might naturally accommodate existing heuristics	- Is usually problem-dependent - Might be difficult to design a method for a given problem
Indirect-constraint handling	- Can be easy to apply many problems (i.e., is not problem-dependent) - Reduces problem to simple optimization - Allows user preferences by weights	- Requires many penalty parameters - Requires prior knowledge of degree of constraint violation - Does not contribute computational efficiency (evaluate all solutions)

Table 2.6 Constraint Handling Approaches in Evolutionary Algorithms

Approach		References
Direct constraint handling	Elimination	Michalewicz and Xiao (1995); etc.
	Repairing	Liepins and Vose (1990); Liepins and Potter (1991); Michalewicz and Janikow (1991); Nakano (1991); Muhlenbein (1992); Orvosh and Davis (1993 and 1994); Le Riche and Haftka (1994); Michalewicz and Xiao (1995); Tate and Smith (1995); Xiao and Michalewicz (1996 and 1997); Steele et al. (1998); etc.
	Preserving	Davis (1991); Michalewicz and Janikow (1991); Michalewicz et al. (1991); Michalewicz (1996); Whitley (2000); etc.
	Decoding	Palmer and Kershenbaum (1994); Dasgupta and Michalewicz (1997); Kim and Husbands (1997 and 1998); Kowalczyk (1997); Koziel and Michalewicz (1998); etc.
	Locating boundary of feasible regions	Schoenauer and Michalewicz (1996 and 1998); etc.
Indirect-constraint handling (Penalty method)	Death penalty	Schwefel (1981); Back et al. (1991); etc.
	Static penalty	Richardson et al. (1989); Goldberg (1989); Back and Khuri (1994); Homaifar et al. (1994); Huang et al. (1994); Olsen (1994); Thangiah (1995); Le Riche et al. (1995); Morales and Quezada (1998); etc.
	Dynamic penalty	Joines and Houck (1994); Michalewicz (1995); Kazarlis and Petridis (1998); etc.
	Annealing penalty	Michalewicz and Attia (1994); Carlson and Shonkwiler (1998); etc.
	Adaptive penalty	Hadj-Alouane and Bean (1992); Smith and Tate (1995); Yokota et al. (1996); Gen and Cheng (1996); Eiben and Hauw (1998); etc.
	Co-evolutionary penalty	Coello (2000)

2.3.1 Direct Constraint Handling

Elimination Method

Elimination methods, which are also known as abortion methods (Michalewicz and Michalewicz, 1995)[8], are employed to remove infeasible solutions form the population. Such methods are aimed for avoiding evaluation of fitness values of infeasible solutions which are possibly generated from a GA. Thus, no infeasible solutions are allowed to be in the population although they are generated from genetic operators embedded in the GA. Elimination is a popular option in many GA applications. However, it has two major drawbacks. First, the elimination method does not allow to the search any chance to traverse on infeasible part of the search space. However, as stated in Michalewicz and Michalewicz (1995), "quite often the system can reach the optimal solutions by crossing an infeasible region especially in non-convex feasible search spaces". Thus, only using elimination methods as constraint handling methods of a GA application should be prohibited. Instead, some combination of other methods (such as repairing, decoding or penalty approaches) would be preferable. Another drawback of the method is that it is usually problem-dependent so that a specific elimination procedure is needed for every particular problem.

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[8] Another well-known classification scheme of the constraint handling techniques for EAs is that of Michalewicz and Michalewicz (1995). They distinguish "pro-life" and "pro-choice" approaches, where "pro-life" methods allow the presence of infeasible solutions in the population, while "pro-choice" approaches disallow it. "Pro-life" covers penalty and repairing methods, while elimination (abortion), decoding, and preserving methods can be classified into "pro-choice" category.

Repairing Method

The repairing method is another popular method used in EAs. The main concept of this method is a combination of learning and evolution processes (Whitley et al. 1994). Through the iterative learning process (e.g., local search for the closest feasible solution), an infeasible solution can be repaired with improved objective value. Note that this method allows presence of infeasible solutions in the population on the contrary to the elimination method. This allows an EA to search infeasible parts of the search space. When an infeasible solution can be easily repaired into a feasible solution, using repairing algorithms may be a good choice for an efficient GA. However, this is not always possible, and in some cases repair algorithms may cause a deterioration in the search process of GAs. Furthermore, for some complex constraint optimization problems (e.g., scheduling and timetable problems), the process of repairing infeasible solutions might be as complex as solving the original problem itself (Michalewicz and Michalewicz, 1995). Another shortcoming of the repair approaches is that there are no standard heuristics for design of repair algorithms since they are also problem-dependent, like elimination methods.

Preserving Method

The main concept of this method is to maintain the feasibility of solutions in the population. Many special operators have been developed for using them as the preserving method. These are partial-mapped crossover (PMX), order crossover (OX), position based crossover (PBX), order-based crossover (OBX), and edge recombination crossover (ERX). They are designed for prohibiting an illogical sequence of genes in offspring which may result from permutation representation with the traditional one-point or two-point crossovers. Note that the preserving approach also has some limitations in its application. The use of the special operators is useful only for the specific application for which they were designed (e.g., project scheduling problem and traveling salesman problem). Application of those operators to a GA in which the genes are represented by real numbers (e.g., XYZ coordinates) rather than binary digits may not be appropriate (this shows that the preserving method is also problem-dependent). In addition, the preserving method requires an initial feasible population, which can pose a hard problem by itself (Craenen et al. 2003). More detailed discussion of the special operators is provided in Goldberg (1996), Gen and Cheng (1997), and Michalewicz (1996).

Decoding Method

The main idea of the decoding method is to transform the original problem (domain of original search space) into another form that is easier to optimize by EAs. This method does not allow generation of infeasible solutions. For instance, a sequence of items for the knapsack problem can be interpreted as a sequence of binary digits ("0" or "1") with an instruction "take an item if possible". By simplifying the problem with an effective decoding method, computation time of GAs can be significantly reduced. Several conditions that must be satisfied when using the decoding method are proposed by Dasgupta and Michalewicz (1997): "(1) for each feasible solution *s* there must be a decoded solution *d*, (2) each decoded solution *d* must correspond to a feasible solution *s*, and (3) all feasible solutions should be represented by the same number of decoding *d*. Additionally, it is reasonable to request that (4) the transformation T_r is computationally fast and (5) it has a locality feature in the sense that small changes in the decoded solution result in small changes in the solution itself". Despite several advantages (see Koziel and Michalewicz, 1998), this method also has some shortcomings. Designing a decoding method for a given problem may be significantly difficult since this method is also problem-dependent; in addition, a transformed problem from a designed decoding method may require more computation time than that required in its original problem.

Locating the Boundary of the Feasible Regions

The main idea in this method is to search areas close to the boundary of the feasible region. According to Coello (2002), the idea was originally proposed in an Operation Research technique known as *strategic oscillation* (Glover, 1977), and has been used in some combinatorial and nonlinear optimization problems (Glover and Kochenberger, 1995). This approach has two basic components: (1) an initialization procedure that is designed for generating feasible solutions, and (2) genetic operators that are employed to explore the feasible region (Coello, 2002). Note that since the approach allows exploring feasible and infeasible regions close to the boundary, a penalty approach may be added to it. The main drawback is that the method is also highly problem-dependent. In addition, it may require complex computation since the feasible regions of the complex optimization problems are usually non-convex and irregular in form. Moreover, there may be several disjoint feasible regions in the problem. Despite such limitations, the approach may be quite efficient and generate good outcomes whenever they are well implemented.

2.3.2 Indirect Constraint Handling (Penalty Approaches)

The penalty method is the most common approach used in EAs (particularly in GAs community) for handling complex constraints. The main idea of this method is to transform a constrained optimization problem into an unconstrained one by adding a certain value (penalty) to the fitness function of the given problem based on the amount (number or severity) of constraint violation presented in a solution.

The penalty should be kept as low as possible, just above the limit below which infeasible solutions are optimal (this is called, the *minimum penalty rule* (Smith and Tate, 1993; Riche et al., 1995)). This should be maintained because an optimization problem might become very difficult for a GA if the penalty is too high or too low. A large penalty discourages GAs exploration to the infeasible regions so as not to move to different feasible regions unless they are very close. On the other hand, if the penalty is not severe enough, then too large a region is searched and much of the search time will be spent exploring the infeasible region due to its negligible impact to the objective function (Smith and Coit, 1997). Such a minimum penalty rule seems simple. However, it is not easy to implement because in many complex problems for which GAs are intended, the exact location of the boundary between the feasible and infeasible regions is unknown (Coello 2002).

Several variations of penalty functions have been developed for handling the infeasible solutions. These are *death penalty, static penalty, dynamic penalty, adaptive penalty, annealing penalty,* and *co-evolutionary penalty*. Yeniay (2005), Coello (2002), and Smith and Coit (1997) extensively reviewed these penalty methods, accounting for advantages and disadvantages of each method. According to their researches, the main problem of most methods is to set appropriate values of the penalty parameters. They suggest that parameter values should be specified based on researchers' good judgments through many experiments. Some famous penalty approaches, which are relatively easy to apply in the GA-based optimization problems, are discussed below.

Death Penalty

The main idea in this method is just to assign a high penalty (i.e., $+\infty$ for minimization problems) when a solution generated from a GA violates any constraint. Therefore, no further calculations are necessary to estimate the degree of infeasibility of the solution. The death penalty method is simple and popular. However, it can perform well only if the feasible search space is not disjointed and constitutes large portion of the whole search space. In addition, if there are no feasible solutions in the initial population (which is normally generated at random), then the evolutionary process will not improve since all the solutions will have the same fitness value (i.e., $+\infty$ for minimization problems). Many studies have reported that the use of this method is not a good choice (Coello 2002; Smith and Coit, 1997). However, it should be noted that the death penalty method can significantly improve the search process of a GA if it works with other efficient constraint handling methods (e.g., a repairing method).

Static Penalty

In this method, simple penalty functions are used to penalize infeasible solutions. The reason why this method is called *static penalty* function is that penalty parameters in the function are not dependent on the current generation number. Two variations on this simple penalty method; one is constant static penalty and the other is a metric-based penalty function. The former method is to assign constant penalty value (P_{cs}) based on the number of constraints that a solution violates regardless of severity of the violations. For instance, if a solution violates n constraints, then the penalty added to the objective function is nP_{cs}. It should be noted that the constant penalty method is generally inferior to the second approach which is based on some distance metric from the feasible region (Goldberg 1989; Richardson et al. 1989). The second approach, which is a more common and more effective penalty method, is to use a soft penalty that includes a distance metric for each constraint, and adds the penalty which becomes more severe with distance from feasibility (Smith and Coit, 1997). A general formulation of this penalty function is as follows:

$$P(\mathbf{j}) = \sum_{i=1}^{m} \eta_i d_i^k$$

where $d_i = \begin{cases} \delta_i g_i(\mathbf{j}), & \text{for } i = 1, \ldots, q \\ |h_i(\mathbf{j})|, & \text{for } i = q+1, \ldots, m \end{cases}$, $\delta_i = \begin{cases} 1, & \text{if constraint } i \text{ is violated} \\ 0, & \text{if constraint } i \text{ is satisfied} \end{cases}$

$\mathbf{j}$ = vector of decision variables

$g(\mathbf{j})$ = function of inequality constraints

$h(\mathbf{j})$ = function equality constraints

q = number of inequality constraints

m-q = number of equality constraints

d_i is the distance metric of constraint i applied to solution $\mathbf{j}$ and k is user defined exponent (normally k=1 or 2). η_i indicates the penalty coefficient corresponding to i^{th} constraint and must be estimated based on the relative scaling of the distance metrics of multiple constraints, on the difficulty of satisfying a constraint, and on the seriousness of constraint violations, or be determined experimentally (Smith and Coit, 1997).

Dynamic Penalty

In the dynamic penalty method, the penalty coefficient (η_i) is usually dependent on the current generation number. Normally the penalty function is defined in such a way that it increases over the successive generations (Coello 2002). The main idea of this method is "allowing highly infeasible solutions early in the search process, while continually increasing the penalty imposed to eventually move the final solution to the feasible region" (Smith and Coit, 1997). A general formulation of a distance based penalty method incorporating a dynamic aspect is as follows:

$$P(\mathbf{j}) = \sum_{i=1}^{m} S_i(t) d_i^k$$

where $S_i(t) =$ a monotonically non-decreasing function

$t =$ number of solutions searched

The primary defect of this method is that it is very sensitive to value of $S_i(t)$, and thus may result in infeasible solutions at the end of evolution. Therefore, this method typically requires problem-specific tuning to perform well (Smith and Coit, 1997). There is no evidence that this dynamic method performs better than the static penalty method.

Adaptive Penalty

The main idea in this method is to reflect a feedback from the search process into a penalty function. Penalty parameters are updated for every generation according to information obtained from the population. There is no general form of this method since it is also highly problem-dependent. Compared to the methods described above, relatively few studies (Hadj-Alouane and Bean 1992; Smith and Tate 1995; Yokota et al. 1996; Gen and Cheng 1996; Eiben and Hauw 1998) have used this penalty method. Readers may refer to Table 2.6 for studies on this subject. Note that some researchers classify this method as a dynamic penalty method.

2.3.3 Summary

Most complex optimization problems have different types of constraints. If GAs are proposed to solve the problems, selection of an appropriate constraint handling method for each constraint is one of the important issues other than developing good genetic operators for the GAs. With the application of good constraint handling techniques, the GAs can effectively solve the problem by guiding the search process away from infeasible solutions.

Various constraint handling techniques used in GAs are reviewed in Section 2.3. They are normally classified into two groups: (i) direct methods (e.g., *elimination, repairing, reserving, decoding, and a method for locating feasible boundaries*) and (ii) indirect methods (assigning a penalty such as *death, static* or *dynamic penalty* to objective function of infeasible solutions). Among them, the most common methods used in GA applications are penalty methods, because they are easy to apply for many complex optimization problems and allow user-specifiable parameters (and weights) in their functional forms. However, normally many parameters are used in the penalty methods, and they must be calibrated from many trials and errors with based on the good judgments of researchers. Furthermore, since the penalty approaches work indirectly in the optimization process (i.e., add penalties to the objective function of the given problem based on the extent of constraint violations, and evaluate all solutions including the infeasible ones), computational burdens may often arise if the problem requires a complex and time-consuming evaluation process.

Other approaches categorized in the direct constraint-handling methods also have shown several advantages although they are highly problem-dependent. They might perform well with GA applications by significantly improving computational efficiency; furthermore, they might naturally accommodate existing heuristics whenever applicable. Thus, familiarity with properties of the given problem is very important, in applying these approaches.

In the GA-based HAO problem, various constraints are specified for: (i) highway design features (e.g., horizontal and vertical curvature constraints) and (ii) geographical and environmental considerations (e.g., environmentally sensitive areas and outside the area of interest). Although the penalty methods can easily handle the solutions violating those constraints, some of them may be more efficiently controlled by the direct methods. For instance, solution alignments, which violate design constraints, may be easily repaired with a simple modification process without evaluating their fitness with penalties. Furthermore, a problem-dependent feasible-boundary approach may be useful for handling solution alignments violating the geographical constraints because good representation of the model constraints can produce good solution alignments during the search process and even reduce computation time. However, since representing complex user preferences and environmentally sensitive areas is also a challenging problem in modeling highway alignments, considerable modeling efforts may be needed to design an appropriate method.

Note that in the proposed optimization model, a GIS is incorporated for the detailed evaluation (right-of-way calculation and environmental impact estimation) of solution alignments generated, and every alignment generated requires massive processing of GIS data during the evaluation process. Therefore, appropriate use of the direct methods (before the GIS evaluation) together with the penalty methods is preferable for efficient handling of infeasible solutions instead of only using the time-consuming penalty approaches. There are many possibilities for using combinations of both the direct and direct methods in GA-based applications.

2.4 Discrete Network Design Problem

The problem we propose in Chapter 1 is quite similar to the network design problem (NDP) which deals with the optimal decisions on the improvement of an existing highway network in response to a growing demand for travel (Gao et al., 2005).

Roughly, this problem can be classified into two different forms: discrete and continuous versions. The discrete version of the problem, known as DNDP, finds optimal (new) highways added to an existing road network among a set of predefined possible new highways while its continuous version, known as CNDP, determines the optimal capacity expansion of existing highways in the network. In whichever form, the objective of the NDP is usually to minimize total system travel cost while accounting for the route choice behaviors of network users. Note that models developed for the CNDP are not reviewed in this dissertation since they are only distantly related to our problem (which considers new highway addition to the existing network).

2.4.1 DNDP with Bi-Level Programming

In many studies (Bruynooghe, 1972; Steenbrink, 1974; LeBlanc, 1975; Johnson et al., 1978; Pearman, 1979; Magnanti and Wong, 1984; Xiong and Schneider, 1992; Yang and Yagar, 1994; Yang and Lam, 1996; Yang and Bell, 1998; Yin, 2000; Lo and Tung, 2001; Meng et al, 2004; Chen and Yang, 2004; Gao et al., 2005; Sharma and Mathew, 2007), the DNDP is usually expressed by a bi-level programming problem in which the upper-level problem represents decision making process of a network designer (e.g., transportation authority), and the lower-level problem represents route choice behavior of the network users under the designer's decision. Note that the bi-level DNDP has also been recognized as one of the most challenging problems in transport (Magnanti and Wong, 1984; Yang and Bell, 1998) due to its computational difficulties; the DNDP is proven to be a NP-complete problem by Johnson et al. (1978).

In the traditional bi-level programming model for the DNDP, it is assumed that the system designers can affect the network users' path-choosing behavior by adding new highways, but cannot control them (i.e., the users make their decision in a user optimal manner). In addition, the traffic demand in the network is assumed to be given and fixed; however, the model allows changes in traffic flow over the network from the improvement of the road network by adding a new highway. A typical formulation of the bi-level programming problem used for the DNDP (Yang and Yagar, 1994) is described as follows:

Upper Level Problem

$$\begin{aligned} &\underset{\mathbf{u}}{\textit{Minimize}} \quad F\left(\mathbf{u},\ \mathbf{v}(\mathbf{u})\right) \\ &\textit{subject to} \quad G\left(\mathbf{u},\ \mathbf{v}(\mathbf{u})\right) \le 0 \end{aligned}$$

where $\mathbf{v}(\mathbf{u})$ is implicitly defined by

Lower Level Problem

$$\begin{aligned} &\underset{\mathbf{x}}{\textit{Minimize}} \quad f\left(\mathbf{u},\ \mathbf{v}\right) \\ &\textit{subject to} \quad \mathbf{g}\left(\mathbf{u},\ \mathbf{v}\right) \le 0 \end{aligned}$$

In the above formulation, F and $\mathbf{u}$ are the objective function and decision vector of upper-level decision makers (system designer) respectively, while G is the constraint set of the upper-level decision vector. f and $\mathbf{v}$ are the objective function and decision vector of lower-level decision makers (users traveling in the network) respectively, while $\mathbf{g}$ is the set of constraints of the lower-level decision vector. It is noted here that $\mathbf{v}(\mathbf{u})$ is implicitly defined by the lower-level problem (i.e., the upper-level objective function F cannot be computed until $\mathbf{v}(\mathbf{u})$ is determined in the lower-level problem).

Upper-Level Problem

In the bi-level DNDP, the upper-level problem, which represents the decision making of the system designer, usually can be formulated as a total cost minimization problem based on the equilibrium traffic flow found in the lower-level problem. Many studies have attempted to solve the upper-level optimization problem in different ways, such as with a decomposition method (Steenbrink, 1974), a Branch and Bound method (LeBlanc, 1975; Poorzahedy and Turnquist, 1982), simulated annealing (SA) based methods (Friesz et al., 1992), genetic algorithm (GA)-based methods (Xiong and Schneider 1992; Yin, 2000; Chen and Yang, 2004; Sharma and Mathew, 2007), and others (Pearman, 1979; Gao et al., 2005 etc.). Among them recently, the GA-based approach is the most popular one because of its simplicity and ability to handle large problems.

Lower-Level Problem

The lower-level problem, which represents the user route choice behavior, can be solved with different types of traffic assignment methods. Choices can be made between static and dynamic assignments, and between deterministic or stochastic assignments. In previous studies on the DNDP, different assignment methods are used for solving the lower level problem. Most studies (LeBlanc, 1975; Friesz et al., 1992; Gao et al., 2005; Sharma and Mathew, 2007; etc.) used the Frank-Wolfe algorithm, which is a deterministic (and static) user equilibrium method, to solve the lower-level problem. In Xiong and Schneider (1992), a neural network approach is used to carry out a deterministic user equilibrium assignment. The stochastic user equilibrium assignment is used in Chen and Alfa (1991), Davis (1994), Lo and Tung (2001), and Meng et al (2004). This dissertation adopts the Frank-Wolfe algorithm to obtain the equilibrium traffic flow pattern.

2.4.2 Summary

Through Section 2.4, the discrete network design problem (DNDP) has been briefly reviewed. The models developed for the DNDP should deal with a relatively larger highway network than those for the alignment optimization problem by considering optimal investment decisions for adding new highways in a given highway network. A conceptual road network with sets of nodes and arcs is used in the DNDP models to represent trip generators (e.g., traffic cities) and new and existing highways. However, such macro-level models may be impractical to use in a real highway construction project directly. In a real world situation, highway planners usually want to determine where to connect a new highway to the existing road network and to design the alignment of the new highway. In addition, there are many other significant factors to be considered in the problem, such as geometric design features and environmental impacts of new highways. In the literature, no model dealing with the DNDP considers costs relevant to highway construction (e.g., earthwork and right-of-way costs) as well as geographic and environmental concerns. The DNDP models just assume a set of possible highways is given as a model input and only consider whether it should be linked in the network with two binary integer values (e.g., "1"=add and "0" not add). Despite such limitations, the main advantage of the DNDP models is to consider the equilibrium traffic flow pattern for estimating user costs of various alternative highways, which is more reasonable than models developed for the alignment optimization problem. Table 2.7 presents basic differences between the DNDP and the HAO problem.

The structure of the bi-level programming problem, which is also used in the DNDP, is suitable for solving the network optimization problem proposed in this dissertation. The upper-level problem of the model structure is the highway alignment optimization (HAO) problem, which optimizes 3D highway alignments, and the lower-level problem is the equilibrium traffic assignment problem.

It should be noted here that the HAO problem itself is a very complex problem requiring time consuming search process; if we also add the traffic assignment process in the problem, its computational burden will increase further. Therefore, development of efficient solution search algorithms is essential for handling this larger problem.

Efforts in computation time reduction of the upper-level problem are covered in Part I of this dissertation by introducing efficient solution search methods. The network version of the highway alignment optimization problem is introduced in Part II.

Table 2.7 Basic Differences between DNDP and HAO Problem

	Highway alignment optimization	Discrete network design problem
Scope & Objective	- Find a 3D highway alignment that minimizes costs associated with highway construction	- Find a network configuration that minimizes network travel cost (normally, travel time cost)
Input	- Geometric data associated with highway design - Spatial data (e.g., topography, land-use, property value) of the study area	- Conceptual road network (sets of nodes and links) - Travel demand: origin-destination (O/D) trip matrix
Solutions	- 3D highway alignments with different cost items and geometries	- Conceptual road networks with different combinations of given straight lines and points
Output	- Optimized 3D highway alignments - Detailed total cost components - Environmental impact summary	- Conceptual road network - Network travel cost
Advantage	- Can generate realistic 3D alignments - Can evaluate numerous candidate-alignments - Can work in continuous search space - Can consider all dominating and alignment sensitive costs - Can exploit massive amounts of information in a GIS with a dynamic link library	- Can reflect drivers' route choice behavior from a traffic assignment process - Can deal with larger networks - Gives a conceptual network frame
Dis-advantage	- Cannot reflect route choice behavior of the network drivers for different highway alternatives in the optimization process - Has great computational burden for a large network	- Cannot consider detailed highway costs and constraints associated highway construction - Cannot consider geographical and environmental features - Cannot generate and evaluate new highway alignments

PART I: COMPUTATIONAL EFFICIENCY AND SOLUTION QUALITY ISSUES IN HIGHWAY ALIGNMENT OPTIMIZATION

Part I introduces two distinct solution search methods developed for optimizing three-dimensional (3D) highway alignments effectively. The integrated GA-GIS based method (Jong and Schonfeld, 2003 and Jha and Schonfeld, 2004) is employed in the proposed optimization model as a base search-method, and (i) the feasible gates (FG) approach (see Chapter 3) and (ii) the prescreening and repairing (P&R) method (see Chapter 4) are developed to improve it in computationally efficient and solution promising ways. Each chapter in Part I also provides an example case-study to show how the developed methods work effectively in the alignment optimization process.

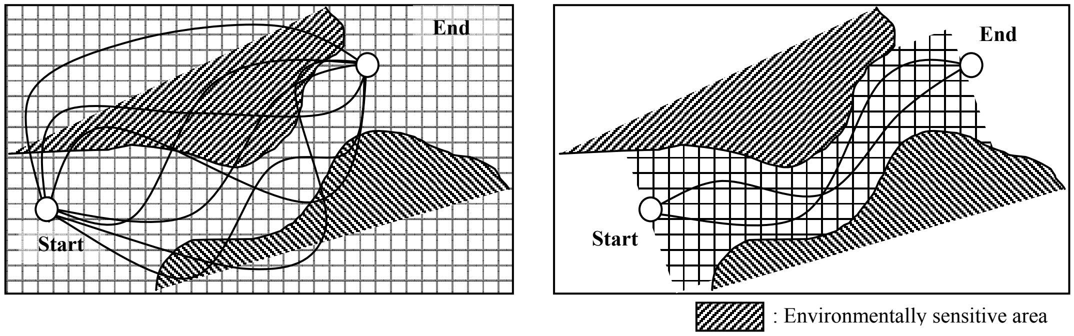
Chapter 3: Highway Alignment Optimization through Feasible Gates

This chapter describes an effective constraint handling method (called feasible gate (FG) approach) developed for improving computation efficiency of the alignment optimization process. The method mainly aims to realistically represent complex geographical (spatial) constraints for the alignment optimization as well as to control solution alignments violating those constraints. A research motivation of the FG method is discussed in Section 3.1, and its methodologies applied for horizontal and vertical alignments are presented in Sections 3.2 and 3.3, respectively. Two example studies presented in Section 3.4 demonstrate the capability of the proposed method. Finally, Section 3.5 summarizes this chapter.

3.1 Research Motivation of Feasible Gate Approach

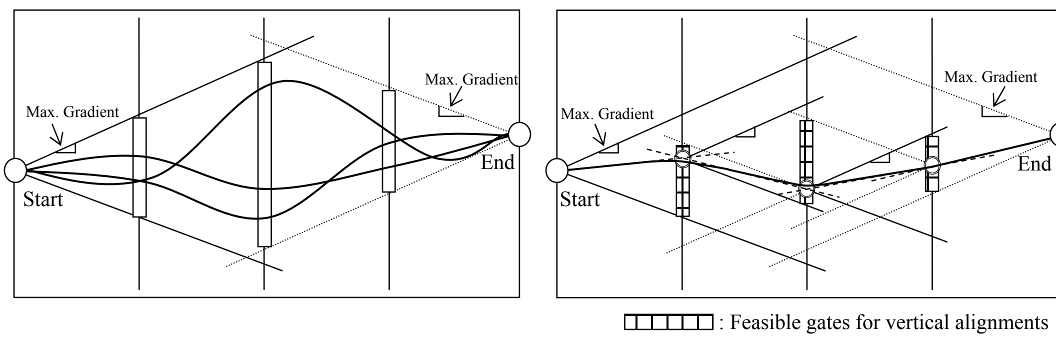
As stated previously in Section 2.1, various costs and factors are associated with in highway alignment design process. Among them, the effects of alignments on environmentally sensitive areas are often regarded as the most attractive ones and complex effects in recent highway construction projects. User preferences including political issues may also be critical in selecting rights-of-way of the alignments. These factors are normally intangible and not easily estimated in monetary values; however, they may greatly reduce the alignment search problem by excluding many possibilities and requiring alignments to pass through some narrow "gates" or "corridors".

Until recently, the previously developed HAO model has relied only on a penalty approach to guide the search toward better solutions. It assigned penalties to the cost functions if the solution alignments violated the corresponding constraints, and eventually screened out the candidate solutions whose constraint violations were significant. However, finding the feasible solutions that satisfy geographical and environmental constraints, which are normally provided in undefined functional forms and are problem-dependent, with only such an indirect constraint handling method is computationally expensive. This is mainly because the model has to spend considerable time for evaluating all generated solutions (including the infeasible solutions) with the penalty method. As shown in Figure 3.1(a), many generated alignments may affect the existing environmentally sensitive areas since the search space is the entire area within the rectangular bounds. Such inefficiency is more severe if the sensitive areas are more complex so that the area of interest is also more complex or narrower. Obviously, the solution alignments that violate the sensitive areas cannot be the best solutions; furthermore, the detailed evaluation of each solution takes considerable time. Thus, a good representation of feasible area of interest is needed. An efficient use of the feasible search area can reduce computation time as well as improve solution qualities during the search process. In the model, the computational improvements are desirable since each candidate alignment requires massive processing of GIS data for its detailed evaluation.



(a) Baseline Horizontal Bounds (b) Specified Horizontal Feasible Bounds

Figure 3.1 Bounded Horizontal Search Space



(a) Baseline Vertical Bounds (b) Specified Vertical Feasible Bounds

Figure 3.2 Bounded Vertical Search Space

In this section, feasible gate (FG) methods (for horizontal (HFG) and vertical (VFG) alignments) are proposed (and named) to ensure that complex preferences and environmental requirements are satisfied efficiently in the search process of the optimization model developed. The proposed approaches are intended to avoid generating infeasible solutions that are outside the acceptable bounds and thus to focus the search on the feasible solutions. Figures 3.1(b) and 3.2(b) provide good insights into the proposed FG approaches for horizontal and vertical alignments, respectively. For both vertical and horizontal alignments, the points of intersections (PI's) are only generated here (randomly, by genetic operators from Jong, 1998) along the limited cutting planes orthogonal to the straight line connecting the start and end points, as shown in Figures 3.4 and 3.8. (More complex backtracking alignments are optimized in Jong, 1998.) The key contribution in this part of of the dissertation is to limit the fraction of the cutting planes within which PI's for alignments can be generated, both horizontally and vertically. These limited "gates" are based on user preferences and environmental factors for horizontal alignments and on allowable gradients for vertical alignments, after adjustments to allow PI's outside feasible regions if the curved alignments at those PI's stay within feasible regions. By avoiding the generation and evaluation of many infeasible alignments outside the feasible regions, the search for optimized solutions is significantly accelerated.

Particularly for horizontal alignments, since various spatial considerations apply, the preferred horizontal feasible gates may be quite complex, discontinuous, and significantly depend on the preferences of model users. Therefore, ways of dealing with the various user preferences and reflecting them in the solution search process are key issues to be resolved. It is relatively easier to ensure feasible gates for vertical alignments than for horizontal ones. The feasible ranges are usually bounded by maximum gradients.

3.2 Feasible Gates for Horizontal Alignments

A horizontal feasible gate (HFG) approach is developed to realistically represent a complex horizontal search space in modeling highway alignments. In addition, since it requires interactive use of the spatial information in the study area, an input data preparation module (IDPM) is also developed. The IDPM is a customized GIS with ArcView GIS 3.x designed for easy preparation of the model inputs. With incorporation of the IDPM into the HFG-based approach, we now enable the model users to interactively specify their preferences (e.g., areas of interest) on given GIS maps and enhance the model solution quality and computation efficiency.

3.2.1 User-Defined Horizontal Feasible Bounds

Figure 3.3 shows how the existing GIS maps and user's areas of interest are converted to the model-readable format through the IDPM. It is noted that digitized land use and property information (e.g., values and boundaries) maps are essential in using IDPM. Since GIS databases are widely used, some (even property maps) are available nowadays free or with some charges at the USGA, ESRI, and other websites of companies and local governments.

Let *BSA* be a baseline study area in which properties are spatially distributed in a rectangular space and k be a clipped property piece resulting from the superimposition of different map layers (refer to Figure 3.3). Additionally, let U be our area of interest, U' be the area outside it, and E and E' be environmentally sensitive and insensitive areas, respectively. Then U_k denotes whether k is inside U and E_k indicates whether it is inside E. These variables are used to represent horizontal feasible bounds (*HFB*), untouchable areas (*HFB′*), and the set of feasible gates for PI's in the next section.

$$U_k = \begin{cases} 0\text{: If property piece } k \text{ is outside } U \\ 1\text{: If property piece } k \text{ is inside } U \end{cases}, \quad E_k = \begin{cases} 0\text{: If property piece } k \text{ is outside } E \\ 1\text{: If property piece } k \text{ is inside } E \end{cases}$$

$$k \in \begin{cases} HFB & \text{if } U_k\text{=1 and } E_k\text{=0, } \forall k \text{ in } UFB \\ HFB' & Otherwise \end{cases}$$

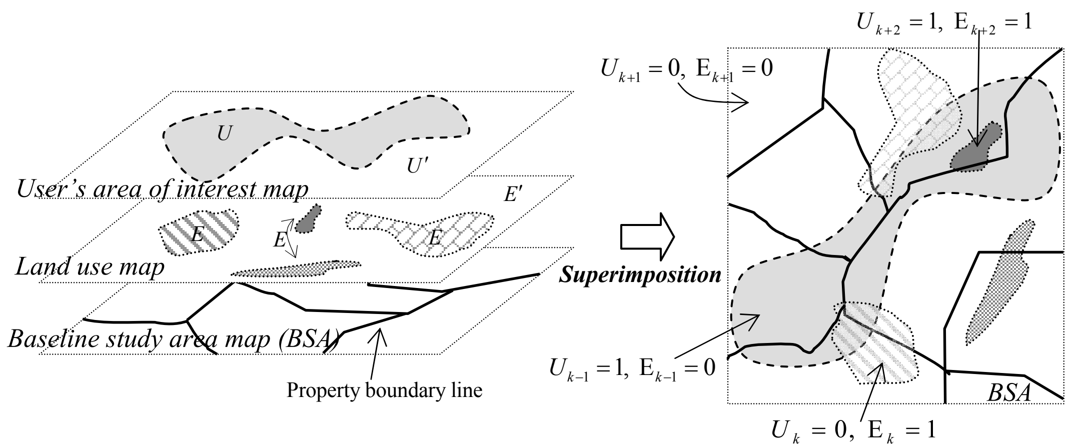


Figure 3.3 Setup of User-Defined Horizontal Feasible Bound with IDPM

3.2.2 Representation of Horizontal Feasible Gates

Let $Start_H = (x_s, y_s)$ and $End_H = (x_e, y_e)$ be horizontal start and end points of a new alignment, and $\overline{SE}$ denotes the line connecting $Start_H$ and End_H. Jong (1998) introduced vertical cutting lines, which are perpendicular to $\overline{SE}$, to find horizontal PI's of the alignment along the cutting lines (perpendicular to the straight line connecting the two end points, as shown in Figure 3.4) in a rectangular search space. We adopt that concept in this study to realistically represent the set of horizontal feasible gates (HFG) with the specified-horizontal feasible bounds (*HFB*) as shown in Figure 3.4. Jong's key variables and equations required to express the proposed method are presented below.

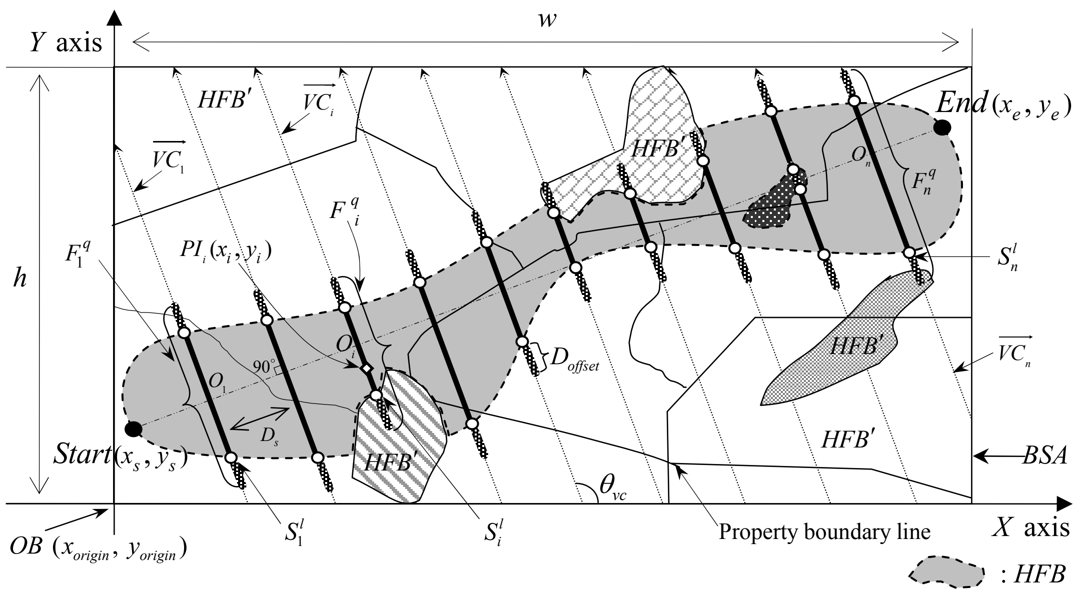


Figure 3.4 Representation of Horizontal Feasible Gates

Suppose that we cut $\overline{SE}$ n times at equal distances between contiguous cuts and let O_i = (x_{oi}, y_{oi}) be the origin of the set of vertical cutting lines, VC_i $\forall i = 1, \ldots, n$. Then the coordinates of O_i are expressed as:

$$\begin{bmatrix} x_{o_i} \\ y_{o_i} \end{bmatrix} = \begin{bmatrix} x_s \\ y_s \end{bmatrix} + \frac{i}{n+1} \begin{bmatrix} x_e - x_s \\ y_e - y_s \end{bmatrix} \tag{3.1}$$

Let θ_{vc} be the angle between the cutting line and the X axis of the given map coordinate system. Then θ can be determined with the following equation:

$$\theta_{vc} = \tan^{-1}((y_n - y_0)/(x_n - x_0)) + \pi/2 \tag{3.2}$$

where $0 < \theta_{vc} < \pi$

We now let $OB = (x_{origin}, y_{origin})$ be the origin of the baseline study area (*BSA)* and h and w be height and width of the study area, respectively. Then the i_{th} vertical cutting line vector, $\overrightarrow{VC_i}$ can be defined as the function of θ_{vc}, O_i, OB, h, and w. Additionally, let d_i be the coordinate of the intersection point at the i_{th} vertical cutting line, and d_{iU} and d_{iL} be its upper and lower bounds, respectively. Then, $\overrightarrow{VC_i}$, d_{iU}, and d_{iL} are determined based on the following four cases:

Case 1: If $\theta_{vc} = 0$ or π

$$\begin{aligned} d_{iU} &= \left(x_{origin} + w\right) - x_{o_i} \\ d_{iL} &= \left(x_{origin} - x_{o_i}\right) \\ \overrightarrow{VC_i} &= \overrightarrow{(x_{origin}, y_{o_i})(x_{origin} + w, y_{o_i})} \end{aligned} \tag{3.3a}$$

Case 2: If $0 < \theta_{vc} < \pi/2$

$$\begin{aligned} d_{iU} &= \min\left\{ \left((x_{origin} + w) - x_{o_i}\right)\big/\cos\theta_{vc},\ \left((y_{origin} + h) - y_{o_i}\right)\big/\sin\theta_{vc} \right\} \\ d_{iL} &= \max\left\{ \left(x_{origin} - x_{o_i}\right)\big/\cos\theta_{vc},\ \left(y_{origin} - y_{o_i}\right)\big/\sin\theta_{vc} \right\} \\ \overrightarrow{VC_i} &= \overrightarrow{(x_{o_i} + d_{iL}\cos\theta_{vc}, y_{o_i} + d_{iL}\sin\theta_{vc})(x_{o_i} + d_{iU}\cos\theta_{vc}, y_{o_i} + d_{iL}\sin\theta_{vc})} \end{aligned} \tag{3.3b}$$

Case 3: If $\theta_{vc} = \pi/2$

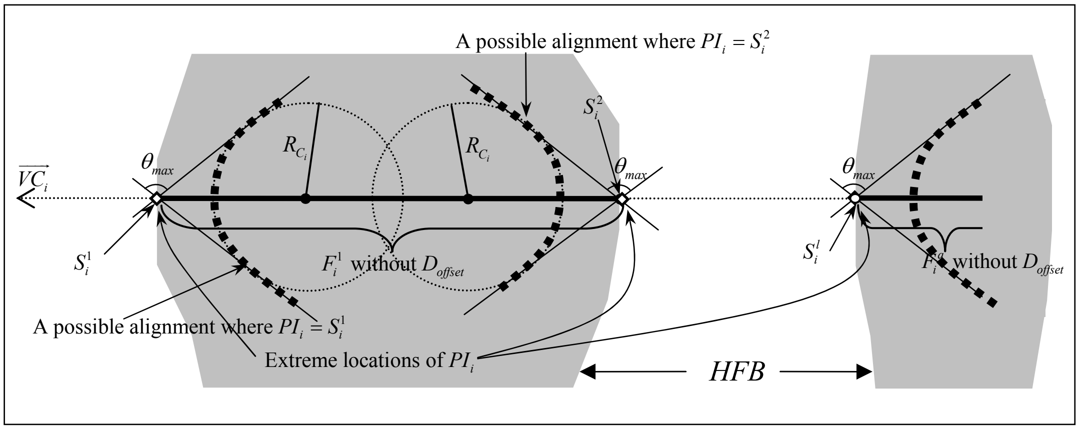
$$\begin{aligned} d_{iU} &= \left(y_{origin} + h\right) - y_{o_i} \\ d_{iL} &= \left(y_{origin} - y_{o_i}\right) \\ \overrightarrow{VC_i} &= \overrightarrow{(x_{o_i}, y_{origin})(x_{o_i}, y_{origin} + h)} \end{aligned} \tag{3.3c}$$

Case 4: If $\pi/2 < \theta_{vc} < \pi$

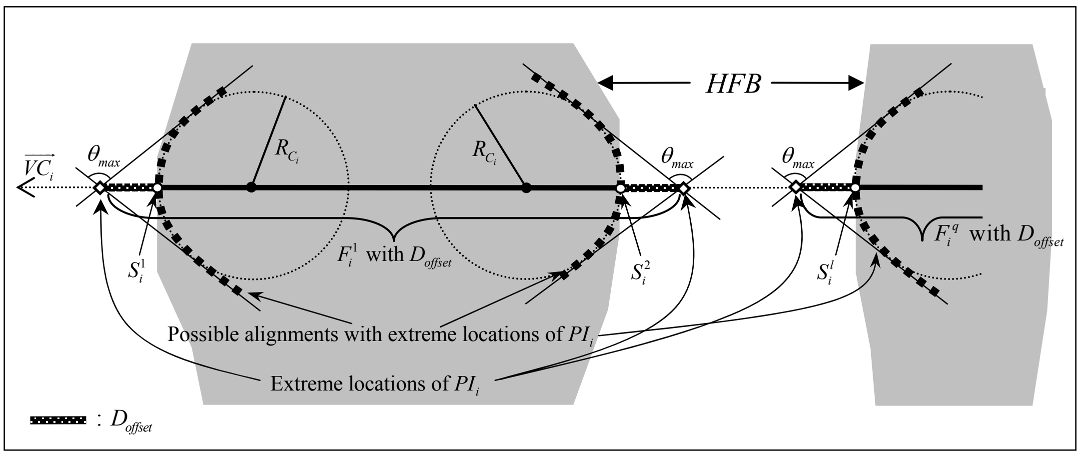
$$\begin{aligned} d_{iU} &= \min\left\{ \left(x_{origin} - x_{o_i}\right) \Big/ \cos\theta_{vc},\ \left((y_{origin} + h) - y_{o_i}\right) \Big/ \sin\theta_{vc} \right\} \\ d_{iL} &= \max\left\{ \left((x_{origin} + w) - x_{o_i}\right) \Big/ \cos\theta_{vc},\ \left(y_{origin} - y_{o_i}\right) \Big/ \sin\theta_{vc} \right\} \\ \overrightarrow{VC_i} &= \overrightarrow{(x_{o_i} + d_{iL}\cos\theta_{vc},\ y_{o_i} + d_{iL}\sin\theta_{vc})(x_{o_i} + d_{iU}\cos\theta_{vc},\ y_{o_i} + d_{iL}\sin\theta_{vc})} \end{aligned} \tag{3.3d}$$

Detailed explanations of the above equations are provided in Jong (1998) and Jong et al. (2000).

Let PI_i be the horizontal point of intersection corresponding to i_{th} vertical cutting line vector ($\overrightarrow{VC_i}$) and S_i^l be the l_{th} intersection point of $\overrightarrow{VC_i}$ with property pieces that are in the specified horizontal bounds (*HFB*) where $l = 1, \ldots, m_i$, and m_i is the total number of intersection points of $\overrightarrow{VC_i}$ with the property pieces in the *HFB*. Then, the q_{th} horizontal feasible gate for PI_i, denoted as F_i^q can be determined by a line segment connecting the two consecutive intersection points (S_i^l and S_i^{l+1}) and an additional allowable offset (denoted by D_{offset}) where $q = 1, \ldots, m_i/2$. As shown in Figure 3.4, the set of horizontal feasible gates F_i^q $\forall i$, $\forall q$ outlines the specified horizontal feasible bound (*HFB*) and is designed to guide the model toward realistic horizontal alignments. The PI's are searched within the specified gates during the model's optimization process and determine the track of the horizontal alignments. Finally, the alignments resulting from the feasible PI's are obtained as candidates to be evaluated with detailed cost components embedded in the model.



(a) Extreme example alignments without D_{offset}



(b) Extreme example alignments with D_{offset}

Figure 3.5 Representation of Allowable Offsets near Horizontal Feasible Gates

The additional allowable offset, D_{offset} is approximately estimated with a simple equation determined by:

$$D_{offset} = R_{C_i} \times \left(1/\cos(\theta_{max}/2) - 1\right) \quad \textit{if there is only a circular curve in the horizontal curved section} \tag{3.4a}$$

$$= \left(R_{C_i} + p_{S_i}\right) \times \sec(\theta_{max}/2) - R_{C_i} \quad \textit{if transition curves are added at both sides of the circular} \tag{3.4b}$$

where R_{C_i} and θ_{max} are the (horizontal) circular-curve radius at PI_i and the maximum allowable deflection angle defined by the model users, respectively as shown in Figure 3.5; note that to compute D_{offset} for a horizontal curved section with transition curves, there need an additional parameter p_{S_i}, which is the offset from the initial tangent to the point of curvature of the shifted circle (refer to Figure 5.13 in Chapter 5).

The allowable offset must be added to the horizontal feasible gates to avoid losing good candidate alignments since it is possible that excellent solutions run near borders between the specified feasible bounds and others, as shown in Figure 3.5(b). Figure 3.5(a) shows a limit of the horizontal feasible gate approach in a case where no allowable offsets ($D_{offsets}$) are provided. Some caution is required in determining the maximum deflection angle in order to fully use of the proposed horizontal feasible gate (HFG) approach. The allowable offset (D_{offset}) becomes excessively long if θ_{max} is too large (e.g., more than $\pi/2$) and R_{C_i} is too long; thus, we hardly expect the benefit of the proposed HFG approach since the long allowable offset may cover the entire length of the vertical cutting line. The minimum curve radius (lower bound of R_{C_i}) is determined by the pre-specified design speed, maximum superelevation, and side friction factor. We summarize the feasible gate determination procedure as follows:

Feasible Gate Determination Procedure (3.1)

STEP 1: for $i = 1$ to n

    find S_i^l and m_i $\leftarrow$ only if $\overrightarrow{VC_i}$ intersects k in HFB $\forall k$ in HFB

  end

STEP 2: for $l = 1$ to m_i

    discard duplicate S_i^l

  end

  update m_i

STEP 3: for $i = 1$ to n

    for $q = 1$ to $m_i / 2$

      for $l = 1$ to m_i

        $F_i^q = \overline{S_i^l \; S_i^{l+1}}$

        add D_{offset} at the both ends of F_i^q

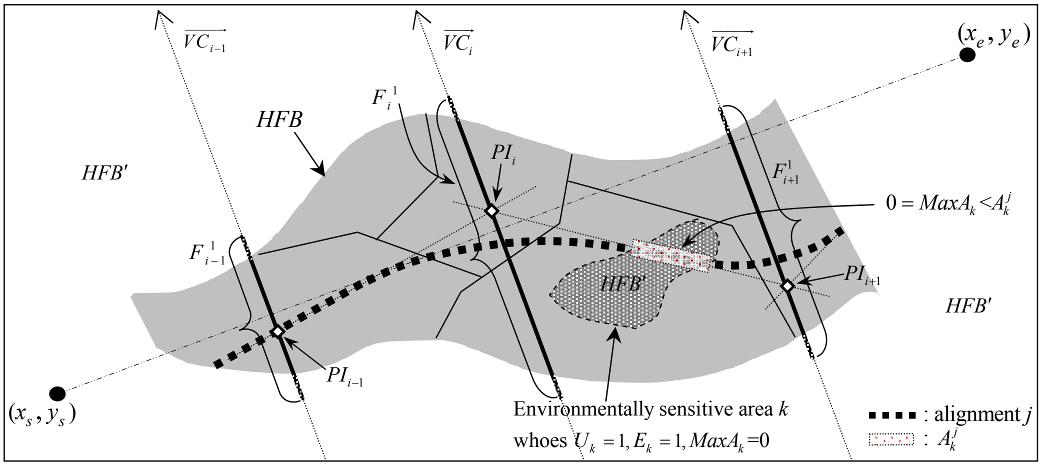
      end

    end

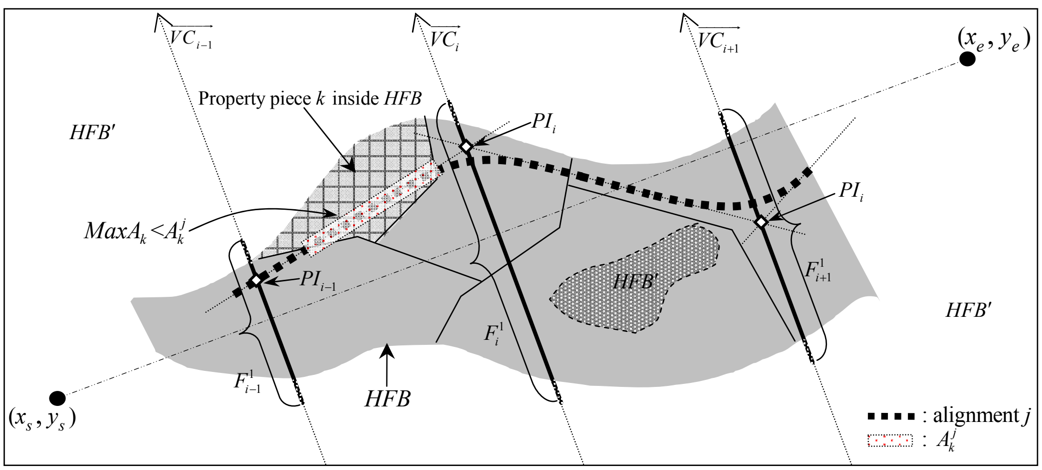
  end

3.2.3 User-Defined Constraints for Guiding Feasible Alignments

We have set the horizontal feasible bound (*HFB*) and represented the feasible gates of PI's for horizontal alignments realistically through Sections 3.2.1 and 3.2.2. It is noted, however, that the derived feasible gates do not always guarantee that feasible alignments are generated which satisfy complex geographical constraints defined by the model users. For instance, solution alignments generated from the optimization model might still affect the untouchable areas (*HFB′*) if they are surrounded by or in the middle of the feasible bounds (*HFB)* as shown in Figure 3.6(a). In addition, the alignments might affect areas of property piece k by more than the allowable amounts specified by the model users, as shown in Figure 3.6(b).



(a) An example alignment affecting the environmentally sensitive area



(b) An example alignment affecting user-specified property piece k with $MaxA_k < A_k^j$

Figure 3.6 Example Alignments Violating User-Defined Constraints

To represent such a problem, we let A_k and $MaxA_k$ be total area of property piece k and its maximum allowable area affected by the alignment, respectively. $MaxA_k$ is initially set to be A_k for the property piece k inside the *HFB*, and be 0 for the property outside it; $MaxA_k$ can be interactively manipulated by the model users with the developed IDPM. A typical penalty function used for dealing with this problem can be expressed as:

$$P_{HG_k} = \begin{cases} \beta_{HG_0} + \beta_{HG_1} \times (A_k^j - MaxA_k)^{\beta_{HG_2}}, & \text{If } MaxA_k < A_k^j \\ 0 &, \quad Otherwise \end{cases} \tag{3.5}$$

where A_k^j = Area of property piece k affected by alignment j

P_{HG_k} = Penalty associated with area of property piece k affected by alignment j

β_{HG_0}, β_{HG_1}, and β_{HG_2} are peanlty parameters for the user-defined geographical constraints

The function, P_{HG}, which is known as a soft penalty function, is widely used in many studies (Jha, 2000; Jha and Schonfeld, 2004). Similar forms are also used in the prescreening and repairing (P&R) method (in Chapter 5) to control the solution alignments which are insufficient to accommodate required curve length. This penalty function is intended to smoothly guide the search in the optimization model. A penalty is assigned to the objective function value of the alignment if it violates the constraints.

Table 3.1 presents spatial attributes of the baseline study area map, and they are created from the IDPM interactively with the model users. Rows shaded in the table represent property pieces in the defined horizontal feasible bound (*HFB*). As stated previously, each property piece k has index variables, U_k and E_k , identifying whether it is inside U and E, respectively. There are unit property cost and area of k (denoted by C_k and A_k, respectively) in the table to calculate the right-of-way cost of the alignment. In addition, $MaxA_k$ and *Land-use* are also included in the attribute table to reflect the user-defined constraints and to estimate environmental impacts of the alignment, respectively.

The proposed horizontal feasible gate (HFG) method can also be applied to the fixed points in which a new alignment intersects with an existing road and stream or user-specified points. Each of those may require different specific constraints. For instance, constraints might limit the number of intersections if an alignment should not intersect an existing highway more than twice. Constraints might also limit the minimum vertical clearance if the alignment should pass over the existing highway. The proposed approach is applicable to many other cases if corresponding GIS data are available.

Table 3.1 Attribute Table of the Study Area Map Created from IDPM

Shape	*k*	U_k	E_k	C_k	A_k	$MaxA_k$	*Land use*
Polygon	1	1	0	0.15	3,504	3,504	Farm
Polygon	2	0	1	0.01	1,000	0	Wetland
Polygon	3	1	0	10.20	2,035	200	Resident
Polygon	4	1	0	11.04	890	100	Resident
Polygon	5	0	0	0.25	4,082	0	Park
Polygon	6	1	1	0.12	1,730	0	Cemetery
Polygon	7	0	0	13.44	2,150	0	Commercial
Polygon	8	0	0	12.63	1,830	0	Resident
Polygon	9	1	0	0.02	1,632	1,632	Stream
Polygon	10	0	1	2.16	1,024	0	Historic
Polygon	11	1	0	0.88	851	100	Historic
Polygon	12	0	1	0.10	3,730	0	Cemetery
⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮

3.3 Feasible Gates for Vertical Alignments

To represent the vertical feasible gates (VFG) of an alignment, we adopt the orthogonal cutting plane method developed in Jong (1998) and Jong and Schonfeld (2003), which is an extension of the vertical cutting line concept to the three-dimensional (3D) alignment optimization. We first let the *HZ* plane be a coordinate system designed to represent ground and road elevation along the horizontal alignment. The *H* and *Z* axes represent road distance and elevation along the horizontal alignment, respectively. We now define a vertical alignment on the *HZ* plane. Let $Start_V = (H_0, Z_0)$ and $End_V = (H_{n+1}, Z_{n+1})$ be start and end points of the vertical alignment, respectively where H_0=0 and Z_0, H_{n+1}, and Z_{n+1} are assumed to be known. Then, the set of vertical points of intersection (denoted as $VPI_i \ \forall i$) can be defined as $VPI_i = (H_i, Z_i)$ as shown in Figure 3.7. The set of the consecutive points generally outlines the track of the vertical alignment, while linking each pair of successive points with a straight line produces a piecewise linear trajectory of the alignment (Jong, 1998). The set of vertical feasible gates for *VPI's*, denoted by V_i $\forall i=1, \ldots, n$ are placed in the orthogonal cutting planes (denoted by OC_i $\forall i=1, \ldots, n$) and bounded by upper and lower bounds, Z_i^{LB} and Z_i^{UB} $\forall i=1, \ldots, n$, respectively. Those bounds are determined with the elevations at the previous and subsequent intersection points and a pre-specified maximum gradient, *Gmax.*

The road elevation determination procedure is summarized below, using notation defined in Table 3.2.

Road Elevation Determination Procedure (3.2)

Given with Z_0, Z_{n+1}, $Gmax$, and H_i , find Z_i (for i=1, …, n)

STEP 1: Calculate $tempL_i^1$ and $tempU_i^1$

$$tempL_i^1 = Z_{i-1} - (H_i - H_{i-1}) \times Gmax / 100$$
$$tempU_i^1 = Z_{i-1} + (H_i - H_{i-1}) \times Gmax / 100$$

STEP 2: Calculate $tempL_i^2$ and $tempU_i^2$

$$tempL_i^2 = Z_{n+1} - (H_{n+1} - H_{i-1}) \times Gmax / 100$$
$$tempU_i^2 = Z_{n+1} + (H_{n+1} - H_{i-1}) \times Gmax / 100$$

STEP 3: Calculate Z_i^{LB} and Z_i^{UB} , and go to either STEP 4-1 or STEP 4-2

$$Z_i^{LB} = \mathrm{Max}(tempL_i^1,\ tempL_i^2)$$
$$= \text{ Maximum between } tempL_i^1 \text{ and } tempL_i^2$$
$$Z_i^{UB} = \mathrm{Min}(tempU_i^1,\ tempU_i^2)$$
$$= \text{Minimum between } tempU_i^1 \text{ and } tempU_i^2$$

STEP 4-1: Find Z_i as close as Z_i^g

Case 1: if $Z_i^g < Z_i^{LB}$

$\rightarrow Z_i = Z_i^{LB}$

Case 2: if $Z_i^{LB} \le Z_i^g \le Z_i^{UB}$

$\rightarrow Z_i = Z_i^g$

Case 3: if $Z_i^g > Z_i^{UB}$

$\rightarrow Z_i = Z_i^{UB}$

STEP 4-2: Find Z_i randomly between Z_i^{LB} and Z_i^{UB}

$\rightarrow Z_i = r_c[Z_i^{LB},\ Z_i^{UB}]$

Note that if the new alignment must pass through a certain point (e.g., a cross-point with an existing road), at which elevation is Z_{cp}, with a minimum vertical clearance (ΔH), its elevation at the point may be found from $Z_i = r_c[Z_{cp} - \Delta H,\ Z_{cp} + \Delta H]$.

Table 3.2 Notation Used for Road Elevation Determination Procedure

Notation	Descriptions
H_i =	H coordinate of VPI_i, for i=1, …, n
Z_i=	Z coordinate at VPI_i, for i=1, …, n
Z_i^g =	Ground elevation at H_i
$Gmax$ =	Maximum gradient (%) defined by the model users
$Start_V$ =	Start point of a vertical alignment; $Start_V = (H_0, Z_0)$ where H_0 and Z_0 are given
End_V =	Endpoint of a vertical alignment; $End_V = (H_{n+1}, Z_{n+1})$ where H_{n+1}, is alignment length and Z_{n+1} is given
$tempL_i^1$ =	Provisional lower bound of Z_i based on Z_{i-1}
$tempL_i^2$ =	Provisional lower bound of Z_i based on Z_{i+1}
$tempU_i^1$ =	Provisional upper bound of Z_i based on Z_{i-1}
$tempU_i^2$ =	Provisional upper bound of Z_i based on Z_{i+1}
Z_i^{LB} =	Lower bound of Z_i
Z_i^{UB} =	Upper bound of Z_i
$r_c[A,\ B]$ =	A random value from a continuous uniform distribution whose domain is within the interval [A, B]

A penalty function used for handling the vertical alignments that violate the maximum gradient-constraints in the previous HAO model is as follows:

$$P_{VG} = \beta_{VG_0} + \beta_{VG_1} \times \left(\left| \frac{Z_{i+1} - Z_i}{H_{i+1} - H_i} \right| \times 100 - Gmax \right)^{\beta_{VG_2}} \tag{3.6}$$

where P_{VG} = Penalty associated with violating the maximum-gradient constraints

β_{VG_0}, β_{VG_1}, and β_{VG_2} are peanlty parameters for the maximum-gradient constraints

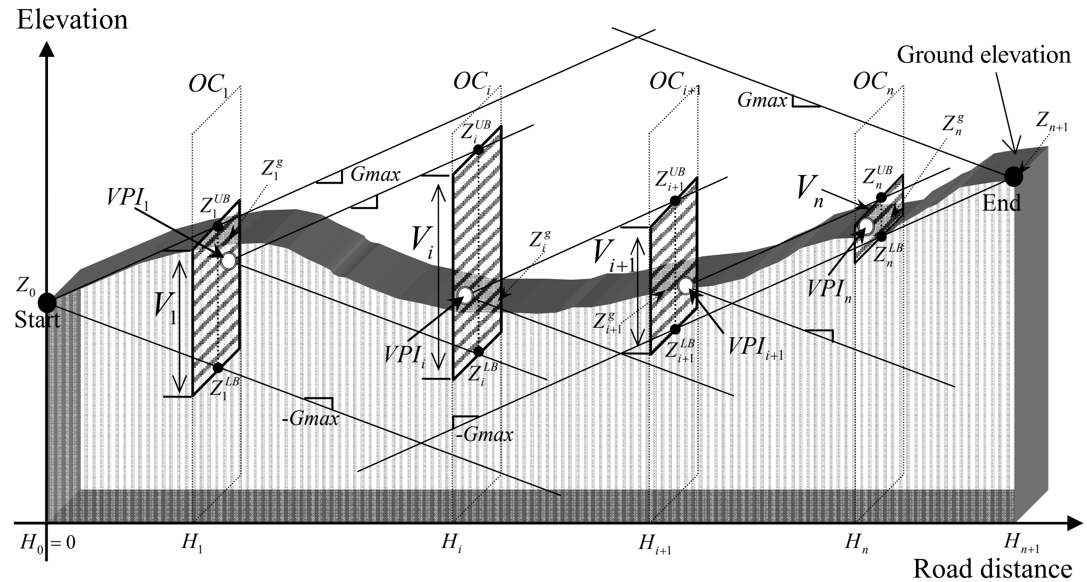


Figure 3.7 Representation of Vertical Feasible Gates

3.4 Example Study

Two example scenarios are tested for the Brookeville Bypass case (on which Chapter 8 provides more information) to demonstrate the performance of the proposed method. One is the solution search with the original search bound in the previous HAO model while the other employs the feasible gate (FG) approach. The baseline major design standards used in this example study are a two-lane road with a 40 foot cross-section (11 feet for lanes and 9 feet for shoulders), a 50 mph design speed, 5% maximum allowable gradient and 6% maximum superelevation. The model runs for 300 generations for each case on a Pentium 4 CPU 3.2GHz with 2GB RAM. The user-specifiable maximum deflection angle, θ_{max} for calculating the allowable offset, D_{offset} is set at π/2 in this example.

To incorporate the horizontal feasible gate (HFG) method in the model, MDProperty View[9] is used as the baseline map and various land-use layers (such as wetlands, historic districts, and residential areas) and a horizontal feasible-boundary map defined by the model users are superimposed on the map. As shown in Figure 3.8, five horizontal feasible gates for PI's realistically represent the user-defined geographical boundary. The allowable offset, which is calculated based on the θ_{max} (= $\pi/2$) and the minimum curve radius for the 50 mph design speed, is added to every feasible gate. Example solution alignments generated with the HFG method are successfully placed within the defined horizontal feasible bound. Note that the 5% maximum allowable gradient is used for determining the vertical feasible gate (VFG) at every VPI in this example.

To test how fast each method (original vs. FG method) finds a reasonable solution, we set a solution boundary based on the optimized solution obtained with 1,000 generations for the same example problem. A "reasonable solution" is defined to be within 2% of the best known solution. Table 3.3 shows that the model tested with the original method finds a reasonable solution in 5,311 seconds (88.52 minutes). However, with the proposed FG method the model finds such a solution in 3,831 seconds (63.85 minutes), with 27.87% savings in computation time. It is noted here that such a computation time saving can significantly be improved if the scale of the road project is enlarged (e.g., the airline distance between endpoints is longer and geographic entities comprised in the study area increase). In this Brookeville example

[9] MDProperty View, developed by Maryland Department of Planning, is "a visually accessible database that allows people to interact with a jurisdiction's property map and parcel information using GIS software." (www.mdp.state.md.us/data)

case, the size of the horizontal study area and the airline distance between two endpoints shown in Figure 3.8 are 3600 feet×8400 feet (1097 meters×2560 meters) and 4,003 feet (1,220 meters), respectively. The study area comprises about 650 geographic entities, including private properties and roads.

Table 3.3 Computation Time Comparison with and without FG Methods for the Brookeville Project

Cases	Original	with FG
Total cost of the solution alignment which first enters the 2% bound of the best-known solution($)*	$4,387,534	$4,387,209
(% of the best solution)	(102.00 %)	(102.00 %)
Program computation time to reach the 2% bound of the best-known solution (sec)	5,311	3,831
Computation time (%)	100.00%	72.13%

* The optimized solution obtained after 1,000 generations (Total cost = $4,301,307) is assummed to be the best.
Note: The model operates on Pentium 4 CPU 3.2GHz with 2GB RAM and considers agency costs only.

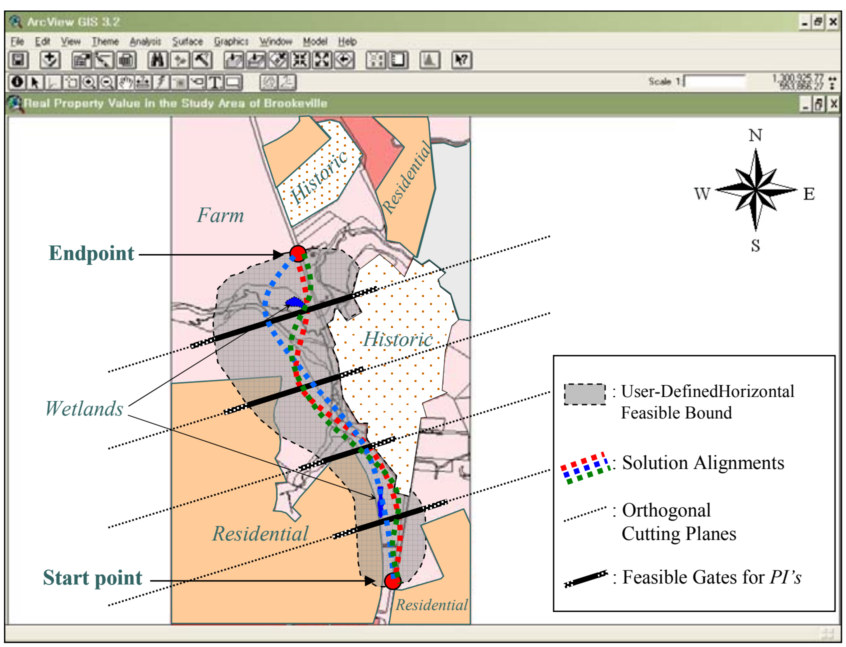


Figure 3.8 Example Solution Alignments Obtained with FG Methods for the Brookeville Project

Figure 3.9 and Table 3.4 show how the solution quality improves over successive generations. With the proposed FG method, the numbers of solution alignments violating the specified constraints, which include the user-defined horizontal and vertical bound constraints (i.e., geographically untouchable and partially untouchable areas and maximum gradient constraints), significantly decrease in early generations as shown in Figure 3.9. About 25% of the solutions with the FG method violate those constraints; however, most solutions with the original method have the constraint violations in early generations. Such an effect can also be found in Table 3.4 showing that total cost breakdowns for the solution alignments at intermediate generations. The solution improvements (i.e., total cost improvements including various cost components) with the proposed method level off earlier than with the original method; the reasonable solution (defined to be within 2% bound of the best known solution) is found between 150 and 200 generations with the FG method, rather than 250 to 300 generations with the original method. This can be interpreted to indicate that the search process in the model now avoids the severely infeasible solutions much sooner and concentrates on refining good solutions with the FG method. With the FG method P_{HG}, which indicates a penalty cost for violating the bound constraints that guide horizontally feasible alignments, slightly affects the total costs of the solution alignments in early generations since the solutions slightly exceed the specified allowable limit of areas; however, the penalty soon disappears in later generations. In addition, P_{VG}, which indicates a penalty cost for violating the bound constraints that guide vertically feasible alignments, does not influence the total cost (P_{VG}=0) through the entire generations since the FG method guides the model to avoid producing vertical alignments outside the feasible gates.

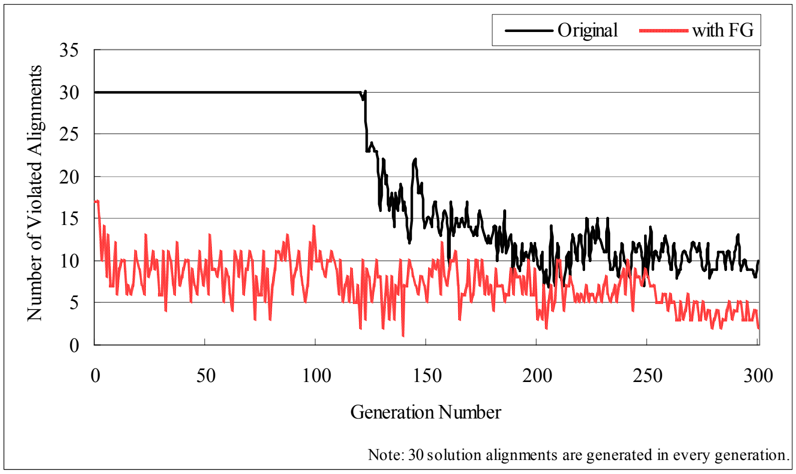


Figure 3.9 Number of Solution Alignments Violating User-Defined Constraints over Successive Generations

Table 3.4 Solution Quality Comparison with and without the FG Methods for the Brookeville Project

Original		Total cost breakdown ($)								Road length (ft)
Gneration #	Total cost ($)	Length-dependent*	Right-Of-Way	Earthwork	Bridge	Grade-separation	P_{HG}**	P_{VG}***	P_D****	
25	1,171,215,706	1,667,371	23,145,300	32,605,080	958,640	30,201	1,043,540,000	69,034,532	234,582	4168
50	344,967,932	1,850,606	55,259	4,984,417	892,925	109,864	235,870,600	101,087,400	116,861	4627
100	342,174,338	1,776,829	52,208	2,312,828	915,010	88,891	235,868,500	101,086,500	73,572	4442
150	6,563,385	1,736,228	49,648	3,678,298	916,665	89,449	48,903	12,048	32,146	4341
200	4,602,704	1,735,343	49,764	1,787,728	922,045	76,993	14,040	7,539	9,252	4338
250	4,435,934	1,734,040	49,851	1,660,253	911,240	76,993	3,557	0	0	4335
300	4,358,150	1,734,920	49,747	1,600,142	906,021	67,320	0	0	0	4337

with FG		Total cost breakdown ($)								Road length (ft)
Gneration #	Total cost ($)	Length-dependent*	Right-Of-Way	Earthwork	Bridge	Grade-separation	P_{HG}**	P_{VG}***	P_D****	
25	6,039,951	1,792,966	52,309	3,011,272	925,805	83,778	10,000	0	163,821	4482
50	4,978,707	1,778,711	51,797	2,015,133	896,290	78,589	11,873	0	146,314	4447
100	4,643,786	1,730,160	49,442	1,872,422	894,305	55,358	20,382	0	21,717	4325
150	4,394,354	1,724,800	49,462	1,612,529	894,470	64,157	5,648	0	43,289	4312
200	4,344,982	1,720,800	49,816	1,599,630	897,162	67,253	0	0	10,321	4302
250	4,328,432	1,720,720	49,459	1,599,586	894,510	64,157	0	0	0	4302
300	4,328,432	1,720,720	49,459	1,599,586	894,510	64,157	0	0	0	4302

* The length-dependent cost represents cost proportional to alignment length including pavement cost and road-superstructure cost.

** Penalty for violating the user-defined horizontal bound constraints (i.e., untouchable and partially untouchable areas within specified limit)

*** Penalty for violating the vertical feasible bound constraints (i.e., ranges of allowable gradients)

**** Penalty for violating other design constraints (e.g., minimum length of vertical curve)

Note: The model runs on Pentium 4 CPU 3.2GHz with 2GB RAM and considers agency costs only.

3.5 Summary

An efficient optimization method called feasible gate (FG) (for horizontal (HFG) and vertical (VFG) alignments) is developed to improve the computation efficiency and solution quality of the alignment optimization process. It improves the search efficiency of the model by restricting the model's search space (horizontally and vertically) so as to maximize the chance that alignments satisfying certain environmental, user preferences and geometric constraints are generated. This is achieved by generating points of intersection (PI's) for alignments only within some appropriately limited subsets ("gates") of the orthogonal cutting planes. A customized GIS module (IDPM) is also developed for integrating the proposed method and the HAO model.

Two test examples with a real road project show how the proposed method improves the model's solution quality and reduces its computation time. Through a realistic application of the model with the FG method, it is found that the model's computation time is reduced by approximately 28%, as shown in Table 3.3, and its solution quality is improved throughout the search process, as shown in Figure 3.9 and Table 3.4. It is noted that the improvement due to the FG method can significantly increase if the scale of the road project is enlarged (e.g., if the number of geographic entities in the study area increases).

The proposed model can now represent a complex road project more realistically and evaluate numerous alignments that satisfy various user preferences since the FG method assists the model in narrowing its horizontal and vertical feasible bounds based on the specified conditions including user preferences. Thus, it can focus sooner on refining the feasible alignments and provides the optimized solutions much faster. The proposed FG approach is expected to be especially applicable in improving existing roads, such as by widening them within very limited bounds, besides optimizing completely new alignments.

Some caution is required in using the FG method. The effect of the FG method would be negligible if the allowable offset (D_{offset}) added to the horizontal gates is excessive; i.e., the horizontal feasible gates for PI's might cover the entire search space of original method if the offset is excessive. On the other hand, it is possible to lose good candidate alignments if the offset is too short. (Excellent solutions may run near borders between the specified feasible bounds and others, as shown in Figure 3.5(b).)

Chapter 4: Prescreening and Repairing in Highway Alignment Optimization

Another efficient solution search algorithm (prescreening and repairing (P&R)), which is also developed for enhancing the computation efficiency and solution quality of the proposed model, is introduced in this chapter. The key idea of this method is to repair (before the detailed evaluation) any candidate alignment whose violations of applicable constraints (here mainly design constraints) can be fixed with reasonable modifications, but discard that alignment (by using a penalty method) and avoid the detailed evaluation procedure if its violations of constraints are too severe to repair. This chapter starts with a research motivation of the P&R method recognized from applying the model to a real highway project. In Section 4.2), the methodology of the proposed method is described, and two examples for the real highway project are tested in Section 4.3 to demonstrate the capability of the method. Finally, Section 4.4 summarizes the results.

4.1 Research Motivation for the P&R Approach

Table 4.1 shows the computation time associated with various cost components in the previous version of the HAO model with a solution result from the Brookeville Bypass project (Kang et al., 2005). The model was tested on Pentium-4 (CPU 3.2 GHZ with 2 GB RAM), and considered only the agency costs (e.g., earthwork cost and right-of-way cost) while suppressing the user costs from the objective function.

Table 4.1 Cost and Computation Breakdowns of an Optimized Solution for the Brookeville Project

Total Cost Components	Cost Breakdown		Computation Breakdown	
	($)	(%)	(sec)	(%)
Right-Of-Way*	49,747	1%	6,300	99.98%
Length-dependent**	1,734,920	40%	0	0.00%
Earthwork	1,600,142	37%	1	0.02%
Grade separation (for a existing road)	67,320	2%	0	0.00%
Bridgde (for a river)	906,021	21%	0	0.00%
Penalty	0	0%	0	0.00%
Total Cost	4,358,150	100%	6,301	100.00%
Total Cost Evaluation Time			6,301	99.15%
Road Generation Time (Horizontal and Vertical Alignments)			54	0.85%
Total Program Computation Time			6,355	100.00%
GIS Program			6,300	99.13%
C Program			55	0.87%
Total Program Running Time			6,355	100.00%

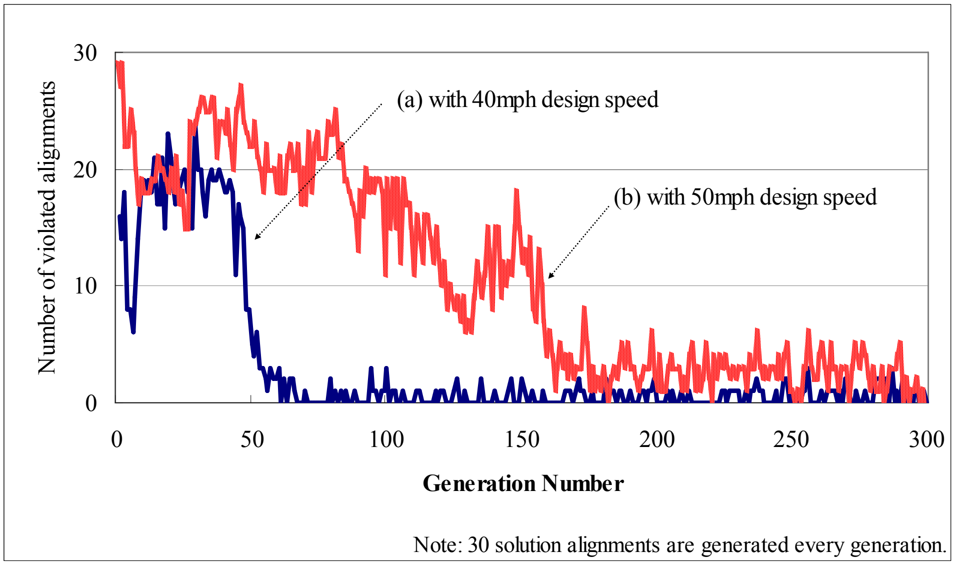
* Right-of-way cost is calculated from GIS.

** The length-dependent cost represents cost proportional to alignment length including pavement cost and road-superstructure cost.

Note: The model runs on Pentium 4 CPU 3.2GHz with 2GB RAM and considers agency costs only.

Basically, the model consists of an optimization module coded in the C programming language and a GIS module. As shown in Table 4.1, the GIS module takes about 99.98% of the total evaluation time; furthermore, it uses 99.13% of total program running time although it only calculates the right-of-way cost and environmental impacts of the alignment; note that the GIS computation time increases significantly if the number of geographic entities in the study area increases. The evaluation time for other cost components is negligible (almost zero), and the alignment generation time is also insignificant (only 0.85% (54 second for this case) of total program running time). Using an artificial grid network for the study area instead of a real GIS map may be a possible way to speed up the program computation time. However, the GIS is crucial in the alignment optimization since it provides the environmental impacts of a new alignment, which are considered to be very important in real roadway projects nowadays. For instance, it computes the areas that the alignment affects among the environmentally sensitive regions (such as, wetlands, residential areas, and historic districts).

Figure 4.1 illustrates how many solution alignments generated by the model violate the design standards (recommended from AASHTO, 2001) associated with design speeds (such as, minimum horizontal curve radius and minimum length of vertical curves) over successive generations for the Brookeville Bypass case. As shown in the figure, many solution alignments violate the design standards in early generations, but their fraction tends to decrease over successive generations. In, addition, the fraction is higher and more persistent when higher (i.e., more restrictive) design standards (e.g., design speeds) are applied. This indicates that many candidate solutions generated by the model (with the GAs) are not feasible, and increasingly so if the required design standards are more constraining. As stated earlier (in Section 2.2.3), the previous version of the HAO model employs a penalty approach to guide the search toward better solution alignments. The alignments are created by fitting curves to tangents connecting a set of PI's generated, and then penalties are assigned to the objective functions of the alignments if they violate the design constraints. Obviously, the infeasible alignments cannot be good solutions; furthermore, the time required for evaluating those alignments takes considerable time (mainly because of the GIS evaluation). Thus, applying the detailed evaluation procedure to all generated solutions by the model is computationally inefficient. That is why the prescreening and repairing (P&R) method is developed and incorporated in the model.



The model runs 300 generations on Pentium 4 CPU 3.2GHz with 2GB RAM for each design speed input

Figure 4.1 Number of Solution Alignments Violating Design Constraints over Successive Generations for Different Design Speeds in the Brookeville Project

4.2 P&R Approach for Violations of Design Constraints

In the HAO model, coordinates (x,y,z) of PI's are randomly created along the corresponding orthogonal cutting planes (refer to Section 2.2.3 or Jong, 1998). Circular horizontal curves and parabolic vertical curves are then fitted; horizontal transition curves can be added in optional. The curve fitting process, originally developed from Jong (1998), follows immediately after series of PI's and resulting tangents between those points are obtained. Ideally, a tangent section must be long enough to contain the required curve lengths which are determined with a design speed and deflection angle at PI's. However, in the original curve fitting process, the curve lengths at both ends are reduced to preserve a continuous alignment if a tangent is insufficient to accommodate the required curve lengths; thus, the resulting alignments from the curve fitting process may not satisfy the design standards.

4.2.1 Basic Concept of P&R Method

In the previous version of the HAO model, penalty functions are only used to control the infeasible alignments during the optimization process. However, it is inefficient and unnecessary to perform the detailed (but time-consuming) evaluation process on the alignments that heavily violate design constraints. The main concept of the P&R method is to find the infeasible segments along a candidate alignment and to repair that alignment before subjecting it to detailed evaluation, thus improving computation time and solution quality. If design constraint violations are detected along the alignment, the P&R method is applied to fix them by shifting the location of the corresponding PI's of the infeasible segments before any detailed evaluation procedure, but skip that evaluation procedure if the violations are too severe to repair. Figure 4.2 shows the concept of the P&R method.

The overall degree of design constraint violation in a candidate alignment can be represented by the percentage of the infeasible curve segments or by that of their corresponding PI's among the total designed PI's. We introduce a parameter, denoted as F_{pr}, to determine whether to repair the infeasible alignment and allow the model users to specify its value.

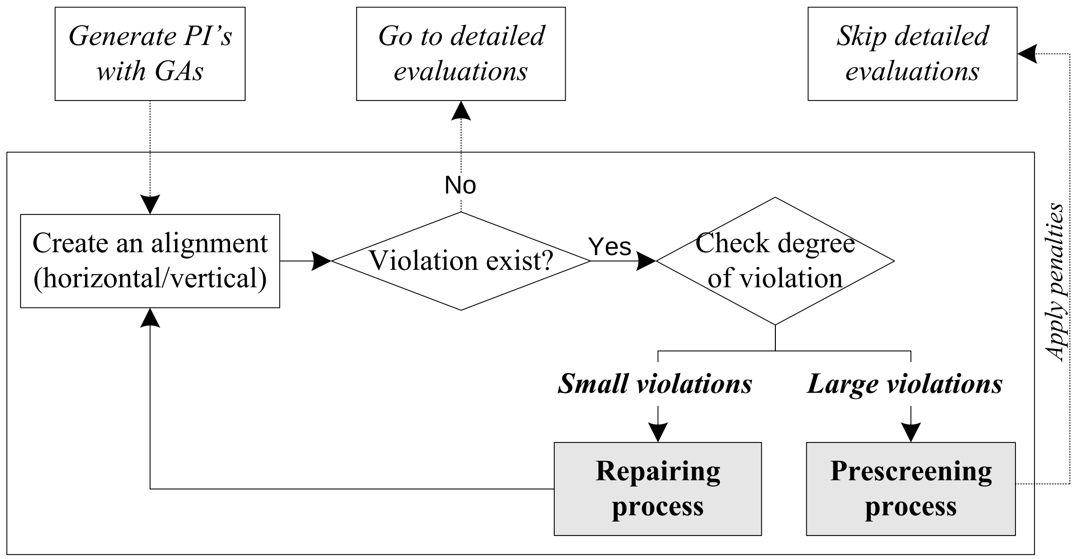


Figure 4.2 Basic Concept of Prescreening & Repairing Method

In the example study of the next section, we use F_{pr} = 50% as a threshold value for distinguishing large violations from small ones. For instance, if 10 horizontal PI's are designed to create the solution alignments of the model, those alignments with more than 6 PI's corresponding to infeasible segments are classified as large violations and assign a penalty to their objective function values. In addition, a distance metric from the satisfied condition (denoted as $Df_i^h < 0$ and $Df_i^v < 0$ in the next section for horizontal and vertical alignments, respectively) is also used for measuring the degree of violations. After incorporation of the P&R method, the model repeats the repairing process until the alignments are fixed; however, if a violation is large, the infeasible solution alignment will be eliminated from the population with its penalty, while the detailed evaluation procedure is skipped. Figure 4.3 shows how the P&R method for the horizontal alignment proceeds.

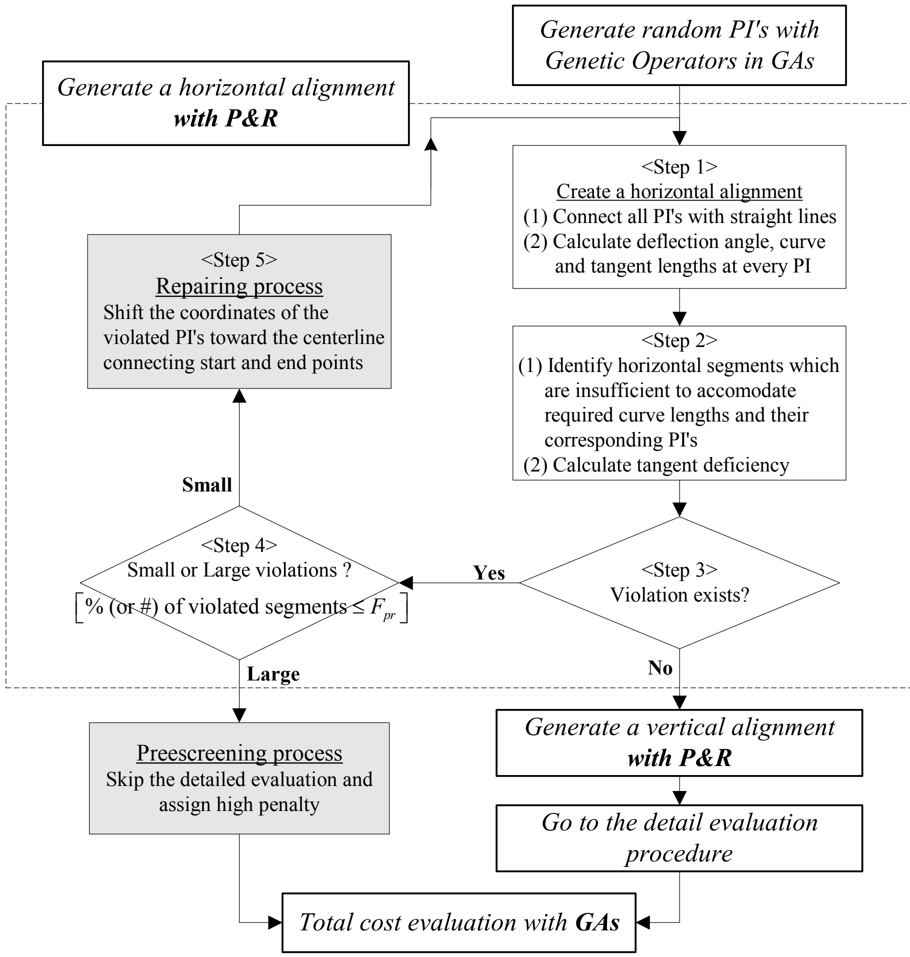


Figure 4.3 Prescreening and Repairing Procedure in Alignment Optimization

4.2.2 Determination of Design Constraint Violations

Let $\mathbf{PI}_i = (x_{pi}, y_{pi})$, be i^{th} horizontal point of intersection and Df_i^h be horizontal tangent deficiency at between $\mathbf{PI}_i$ and $\mathbf{PI}_{i+1}$; note that Df_i^h is used for calculating the significance of the curve fitting violation and is computed as:

$$Df_i^h = \left[L_{PC(i)} + L_{PC(i+1)}\right] - \left\|\mathbf{PI}_{i+1} - \mathbf{PI}_i\right\| \quad \textit{if there is only a circular curve in the horizontal curved section} \tag{4.1a}$$

$$= \left[L_{ST(i)} + L_{TS(i+1)}\right] - \left\|\mathbf{PI}_{i+1} - \mathbf{PI}_i\right\| \quad \textit{if transition curves are added to the circular curve} \tag{4.1b}$$

where $L_{PC(i)}$ = Tangent distance from point of circular curve ($\mathbf{PC}_i$) to $\mathbf{PI}_i$

$\| \ \|$ = Norm (or length) of a vector, and

$L_{ST(i)}$ = Tangent distance from $\mathbf{PI}_i$ to the endpoint of transition curve ($\mathbf{ST}_i$)

$L_{TS(i)}$ = Tangent distance from the beginning of transition curve ($\mathbf{TS}_i$) to $\mathbf{PI}_i$

Note that the following simple equation can be used to compute the tangent distance ($L_{PC(i)}$) when there are no transition curves in a horizontal curved section of the alignment:

$$L_{PC(i)} = R_{C_i} \times \tan\left(\frac{\theta_{PI_i}}{2}\right) \tag{4.2}$$

where R_{C_i} = Radius of the circular curve at $\mathbf{PI}_i$; $R_{C_i} \geq R_m$

R_m = Minimum curve radius based on design speed specified

θ_{PI_i} = Deflection angle at $\mathbf{PI}_i$

For computing tangent distance ($L_{TS(i)}$) when transition curves are added to the circular curve of the horizontal curved section, equation (5.41) in Section 5.2 can be used. The readers may also refer to equation (5.38) in the same section to calculate the deflection angle (θ_{PI_i}) at $\mathbf{PI}_i$.

Now, suppose that a infeasible horizontal curve segment, where the tangent deficiency is greater than zero (i.e., $Df_i^h > 0$), is identified on a solution alignment and its degree of design constraint violation is not huge (i.e., percentage of the infeasible curve segments ≤ F_{pr}), as shown in Figure 4.4. Then, the infeasible alignment can be easily repaired by adjusting either $\mathbf{PI}_i$ or $\mathbf{PI}_{i+1}$ so that the tangent deficiency (Df_i^h) is finally nullified. Which PI should be shifted between ($\mathbf{PI}_i$ and

PI$_{i+1}$) may be determined based on the magnitudes of deflection angles at those PI's. For instance, to repair the infeasible segment shown in Figure 4.4, **PI**$_i$ is kept adjusted (shifted) at the beginning of the repairing process since its deflection angle (θ_{PI_i}) exceeds that ($\theta_{PI_{i+1}}$) of **PI**$_{i+1}$. However, **PI**$_{i+1}$ can also be selected to be adjusted during the repairing iteration whenever its deflection becomes greater than that of **PI**$_i$. The iteration terminates if the infeasible segment is completely repaired.

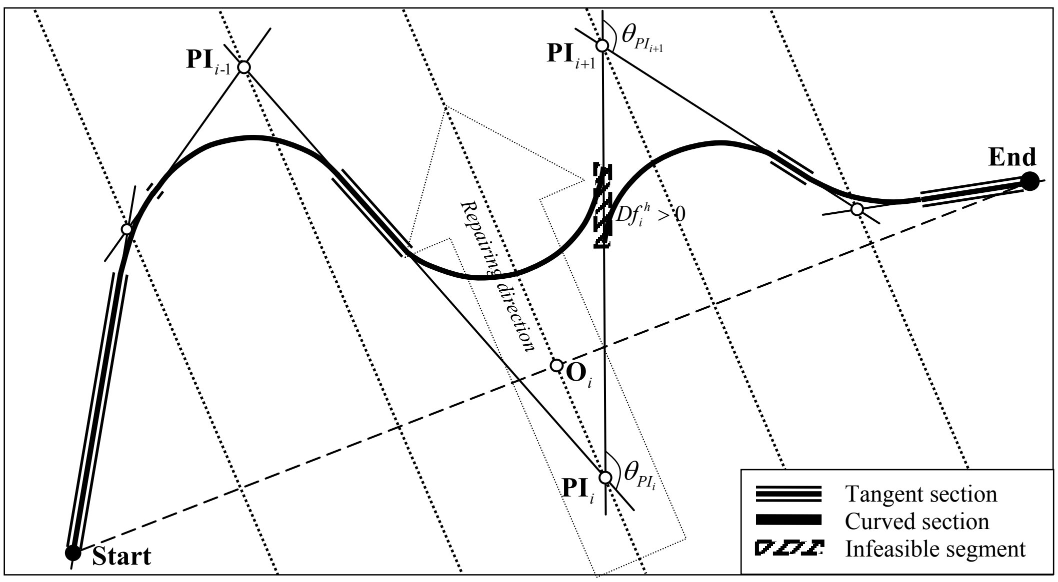
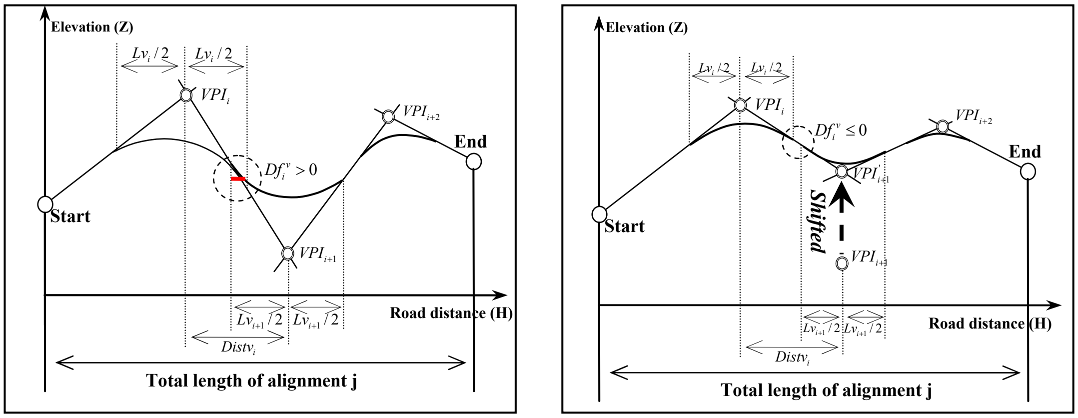


Figure 4.4 Repairing Process for Horizontal Alignments

The P&R method can be almost identically applied for the vertical alignment as in the horizontal alignment. Figure 4.5 show the situation before and after the repairing process for a vertical alignment violating the vertical design constraints.



(a) Before Repairing Process (b) After Repairing Process

Figure 4.5 Repairing Process for Vertical Alignments

Let VPI_i be i_{th} vertical point of intersection (denoted as $VPI_i = (H_i, Z_i)$ in Section 3.3), and Df_i^v be vertical curve-length deficiency at between VPI_i and VPI_{i+1}. Then, Df_i^v can be calculated as:

$$Df_i^v = 1/2(Lv_i + Lv_{i+1}) - Distv_i \tag{4.3}$$

where Lv_i = Vertical curve length at VPI_i

$Distv_i$ = Distance between VPI_i and VPI_{i+1}

In equation (4.3), the vertical curve length (Lv_i) should satisfy minimum sight distance required on crest or sag vertical curve (L_{mi}); i.e., $Lv_i \geq L_{mi}$. The equation used for computing L_{mi} can be found in Jong (1998) or AASHTO (2001). $Distv_i$ can be measured with horizontal distance between VPI_i and VPI_{i+1} on HZ plane (i.e., $Distv_i = H_{i+1} - H_i$).

A soft penalty function (equation (4.4)) is also used in the P&R approach. If an alignment is identified as highly infeasible or if a slightly infeasible alignment cannot be sufficiently repaired (or its violation worsens) during the repairing process, the following penalty function can be used:

$$P_{D^h} = \sum_{i=1}^{n^h} \left[\beta_{D_0} + \beta_{D_1} \times \left(Df_i^h \right)^{\beta_{D2}} \right] \qquad \text{only if } Df_i^h > 0 \tag{4.4a}$$

$$P_{D^v} = \sum_{i=1}^{n^v} \left[\beta_{D_0} + \beta_{D_1} \times \left(Df_i^v \right)^{\beta_{D2}} \right] \qquad \text{only if } Df_i^v > 0 \tag{4.4b}$$

where P_{D^h} = Penalty associated with the tangent deficiency for horizontal alignments

P_{D^v} = Penalty associated with the vertical curve-length deficiency for vertical alignments

n_v = Number horizontal road segments at which $Df_i^h > 0$

n_v = Number of vertical road segments at which $Df_i^v > 0$

β_{D_0}, β_{D_1}, and β_{D_2} are peanlty parameters

With the simple adjustment of the locations of PI's and VPI's, we now can nicely fix the design violations in the solution alignments. Note that the adjusted locations of PI's and VPI's are also random since their previous positions, which are originally obtained from the genetic operators, are randomly distributed along the orthogonal cutting planes.

4.3 Example Study

Two example scenarios are tested for the Brookeville Bypass case (Kang et al., 2005 and 2006) to demonstrate the performance of the proposed method. One is the solution search with the original curve fitting process, and the other is that with the P&R process. The baseline major design standards used in this example study are the same as those used in the case for testing the FG method in Section 3.4.

Recall that we have assumed that the best-known solution is the optimized solution obtained through 1,000 generations for the example project, and "reasonable solutions" are defined as those within 2% bound of the best-known solution. Such tasks are intended to determine how efficiently the model searches for the reasonable solution with proposed P&R method compared to searches with the original curve fitting process.

Table 4.2 Solution Comparisons with and without P&R Methods for the Brookeville Project

Cases	**Original**	**With P&R**
Total cost of the solution alignment which first enters the 2% bound of the best-known solution($)*	$4,387,534	$4,387,270
(% of the best-known solution)	(102.00 %)	(102.00 %)
Program computation time to reach the 2% bound of the best-known solution (sec)	5,311	4,077
Computation time (%)	100.00%	76.77%
Number of alignments that go to the detailed evaluation procedure (%)	8,783 (100.00 %)	8,103 ** (54.32 %)
Number of alignments that skip the detailed evaluation procedure (%)	0 (0.00 %)	6,814 (45.68 %)
Total number of Alignments Generated (%)	8,783 (100.00 %)	→ 14,917 *69.84%* (100.00 %)

* The optimized solution obtained after 1,000 generations (Total cost = $4,301,307) is assummed to be the best.
** Alignments evaluated after being repaired are also included.
Note: The model operates on Pentium 4 CPU 3.2GHz with 2GB RAM and considers agency costs only.

Table 4.2 shows that the model tested with the original curve fitting method finds a reasonable solution in 5,311 seconds (88.52 minutes) while the model with the P&R method finds the solution in 4,077 seconds (67.95 minutes), with 23.23% computation time savings. Such an effect mainly results from the model with the P&R method skipping the detailed evaluation procedure for solution alignments that violate the design constraints; it prohibits the model from exploring the GIS. Furthermore, the proposed method allows the model to consider more alignments than the original case for the same given number of generations (about 69.84% more). However, the original case evaluates all generated alignments including the infeasible alignments; about 36.70% have design constraint violations. As stated previously (see Figure 4.1), the fraction of the infeasible solutions would increase if the design standards were stricter. The P&R approach would become increasingly advantageous as the design standards get higher (i.e., more constraining), since it prescreens and repairs an increasing fraction of the alignments.

Figure 4.6 shows how total cost improves through successive generations for each scenario. For both the cases, most of the improvement is found in the early generations; there is no great improvement of the objective function after about 150 generations. It is noted however that the improvements with the proposed P&R method level off significantly earlier than that with the original curve fitting method. This indicates that with the proposed method the model stays away from the severely infeasible solutions much sooner and concentrates on refining good solutions.

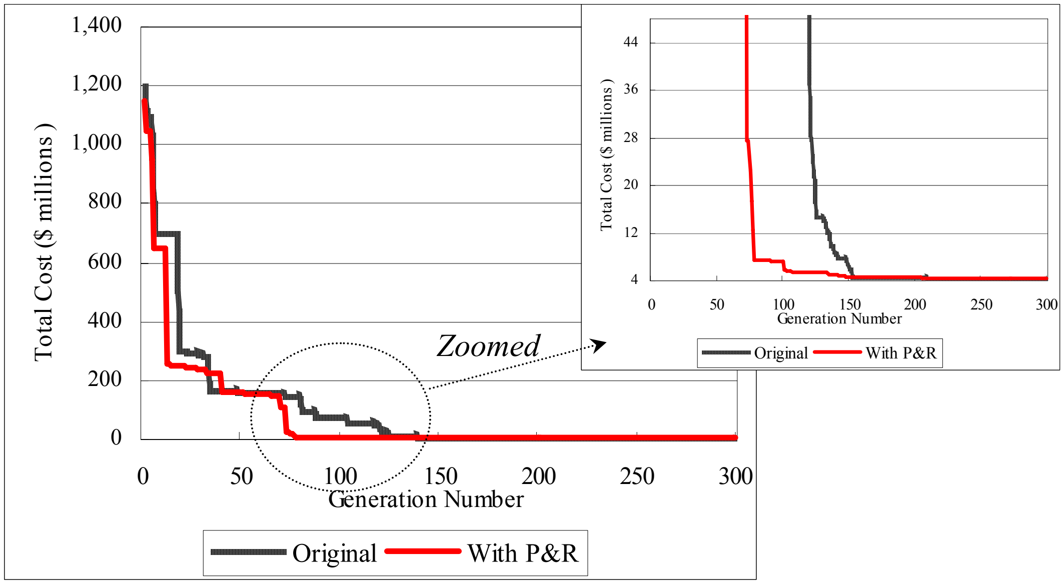


Figure 4.6 Changes in Total Cost over Successive Generations with and without P&R Methods

4.4 Summary

An efficient optimization method (called P&R) is developed to control the design constraints associated in the highway generation procedure. The proposed algorithm is simple, but improves significantly the model's computation time and solution quality. Through the application of the P&R method to a real highway project, its significant contribution to the model computation time is demonstrated. The model can now avoid evaluating the infeasible alignments with its prescreening process and focus on refining feasible alignments with its repairing process. Note that if the P&R method is combined with the FG method proposed in Chapter 3, the computation time for solving the complex highway alignment optimization (HAO) problem would be significantly improved.

PART II: OPTIMIZING SIMPLE HIGHWAY NETWORKS: AN EXTENSION OF HIGHWAY ALIGNMENT OPTIMIZATION

Part II further develops the earlier HAO model, extending its capabilities to alignment optimization for a simple highway network. The model structure is reformulated as a bi-level programming problem; its upper-level problem is formulated as the alignment optimization problem, and (2) lower-level problem is defined as an equilibrium traffic assignment problem.

As stated in Section 1.5 (organization of this dissertation), Part II starts with representations of the highway alignment and its endpoints (see Chapter 5). The basic model formulation and optimization procedure, various cost components (including user cost savings) considered in the model, and a well-known traffic assignment technique together with inputs required for the traffic assignment (e.g., travel time functions and network representation) are discussed in the following chapters.

Chapter 5: Modeling Highway Alignments and Endpoints

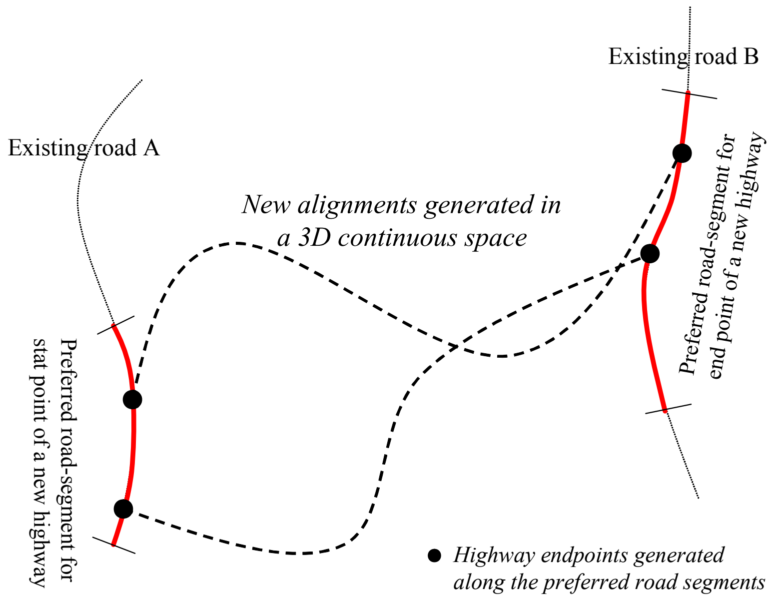


Figure 5.1 Possible New Alignments Connecting the Preferred Road Segments

We assume that drivers' route choice depends on their travel time on the road network. Accordingly, traffic flows on the network including on the new highway alignment may significantly vary depending on its road-length and on where it connects to that network (Figure 5.1).

This chapter realistically represents alignments of the new highway and its endpoints. Highway alignments are realistically described in Section 5.1 with incorporation of transition curves into the highways horizontal curved sections. In Section 5.2, (i) a method is proposed for finding the endpoints of the new highway alignment, and (ii) several types of three-leg cross-structures, which are widely used in the highway engineering, are modeled to realistically represent the endpoints.

5.1 Modeling Highway Alignments

In a highway alignment, a series of tangents and curved sections are adjoined. Circular curves and transition curves are typically combined to form the horizontal curved sections. Some kind of transition curve is often applied between a tangent and a circular curve for mitigating a sudden change in degree of curvature and hence, in lateral acceleration and force, from the tangent to the circular path. Particularly for high-speed highway alignments, spiral transition curves are strongly recommended in horizontal curved sections.

As stated earlier, in the previous version of the HAO model only tangents and circular curves are used to generate the horizontal alignment of a new highway. Through this section, we incorporate transition curves in the curved sections of the horizontal alignments. Such work helps the model produce more realistic alignments during the optimization process. Spiral curves, which are widely used in practice, are chosen to model the transition curves.

5.1.1 Representation of Highway Alignments

Figure 5.2 presents basic segments of a typical horizontal alignment with series of points, representing intersection points between tangents, circular curves, and transition curves. For notational convenience, we let $\mathbf{EP}_1$ and $\mathbf{EP}_2$ be start and end points of a highway alignment respectively, and its initial and final PI's (i.e., $\mathbf{PI}_0$ and $\mathbf{PI}_{n+1}$ respectively) correspond to the start and end points; i.e., $\mathbf{EP}_1$=$\mathbf{PI}_0$ and $\mathbf{EP}_2$=$\mathbf{PI}_{n+1}$ as shown in Figure 5.2. We further use the following notation in Table 5.1 to define the series of points that outline the highway alignment:

Table 5.1 Notation Used for Representing Highway Alignments

Notation	Descriptions
TS$_i$=$\left(x_{TS_i}, y_{TS_i}\right)$	The point of change from tangent to spiral (beginning of spiral) pertaining to **PI**$_i$, $i = 1,\ldots,n$
SC$_i$=$\left(x_{SC_i}, y_{SC_i}\right)$	The point of change from spiral to circle (end of spiral and start of circle at the same time) pertaining to **PI**$_i$, $i = 1,\ldots,n$
CS$_i$=$\left(x_{CS_i}, y_{CS_i}\right)$	The point of change from circle to spiral (end of circle and start of spiral at the same time) pertaining to **PI**$_i$, $i = 1,\ldots,n$
ST$_i$=$\left(x_{ST_i}, y_{ST_i}\right)$	The point of change from spiral to tangent (end of spiral and start of tangent at the same time) pertaining to **PI**$_i$, $i = 1,\ldots,n$

As shown in Figure 5.2, **ST**$_i$ and **TS**$_{i+1}$ are linked by a straight-line section connecting **PI**$_i$ and **PI**$_{i+1}$ for all i = 0,….., n, whereas **TS**$_i$ and **SC**$_i$ and **CS**$_i$ and **ST**$_i$ are connected by a spiral transition curve and **SC**$_i$ and **CS**$_i$ are connected by a circular curve for all i = 1,….., n. Note that in an extreme case, where an alignment tangent section between two consecutive intersection points (e.g., between **PI**$_2$ and **PI**$_3$ in Figure 5.2) is completely eliminated by two spiral transition curves, the point of change from spiral to tangent section pertaining to one intersection point will coincide with the point of change from tangent to spiral curve pertaining to the next intersection point; e.g., **ST**$_2$ and **TS**$_3$ are the same point in Figure 5.2. Furthermore, if an intersection angle at **PI**$_i$ (denoted as θ_{PI_i}) becomes zero, all the reference points pertaining to **PI**$_i$ are the same; for example, the locations of **TS**$_4$, **SC**$_4$, **CS**$_4$, **ST**$_4$, and **PI**$_4$ shown in Figure 5.2 would then all be the same.

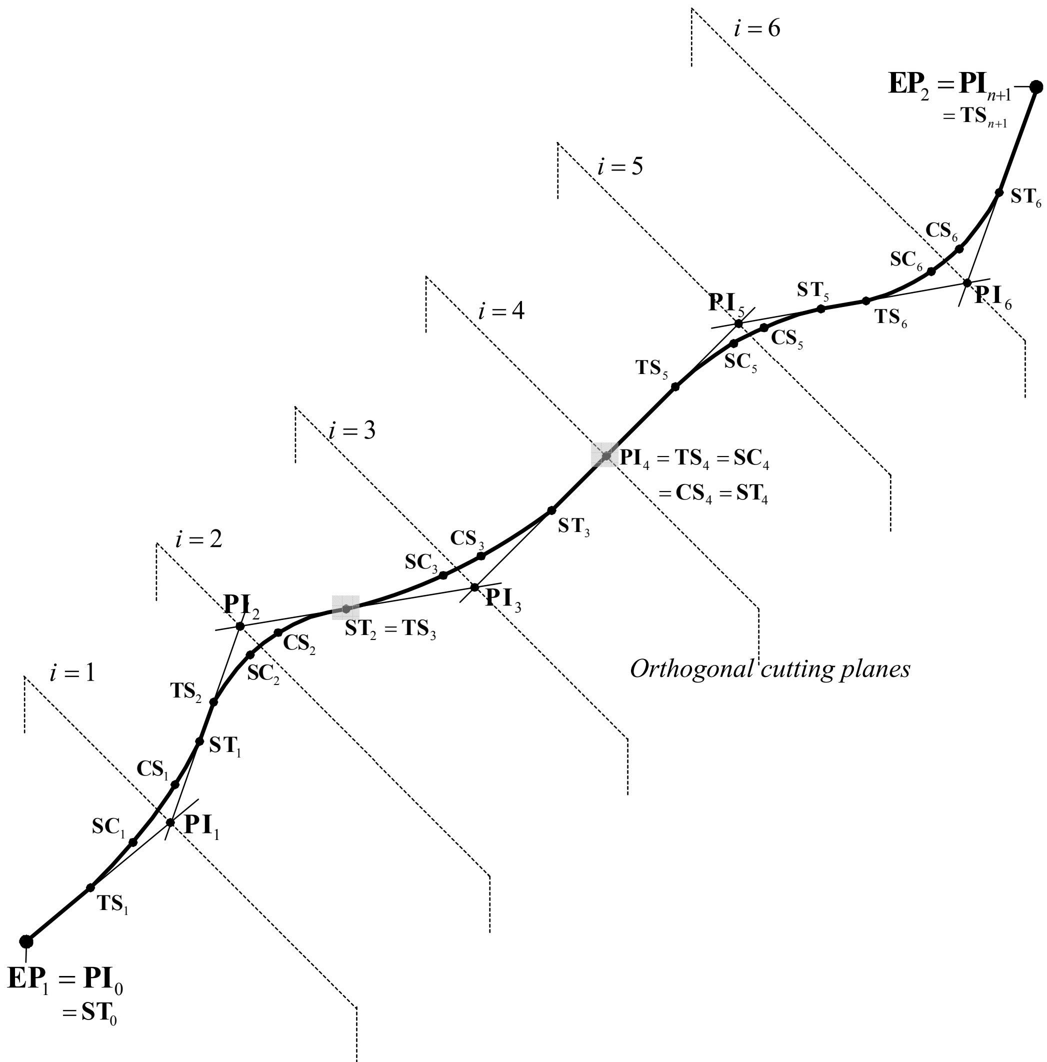


Figure 5.2 Representation of Highway Alignment with Series of Reference Points

We now define the coordinates of all the reference points representing the highway curved section. The coordinates of the highway endpoints, $\mathbf{EP}_1$ and $\mathbf{EP}_2$ can be found through the *endpoint determination procedure (5.2)* in section 5.2.1, and those of the set of $\mathbf{PI}_i$ (for i = 1, …, n) are generated from the customized GA operators (see Jong, 1998) coupled with the FG and P&R methods developed in Chapter 3.

Given such information and design standards required for constructing the highway alignment, the coordinates of **TS**$_i$, **SC**$_i$, **CS**$_i$, and **ST**$_i$ (for i = 1, …, n) can be found with simple vector operations. Figure 5.3 shows the general shape of a horizontal curved section with a circular curve and two spiral transition curves in both sides of the circular. The notation in Table 5.2 is used to describe the curved section:

Table 5.2 Notation and Formulas for Defining Spiral Transition Curves

Notation	Descriptions
$\boldsymbol{\delta}_i =$	The center point of the curved section at **PI**$_i$
$R_{C_i} =$	The radius of the circular curve at between **SC**$_i$ and **CS**$_i$
$\theta_{PI_i} =$	Deflection angle (radian) at **PI**$_i$
$\mathbf{M}_i =$	The middle point of the line segment connecting **TS**$_i$ to **ST**$_i$
$R_{S_i} =$	Variable radius at any point of spiral
$l_{S_i} =$	Spiral arc length from **TS**$_i$ to any point on spiral
$l_{ST_i} =$	Total length of spiral curve from **TS**$_i$ to **SC**$_i$
$\theta_{S_i} =$	Central angle (radian) of spiral arc l_{S_i} ; $\theta_{S_i} = l_{S_i} / \left(2R_{S_i}\right)$
$\theta_{ST_i} =$	Central angle (radian) of spiral arc l_{ST_i} , called "spiral angle"; $\theta_{ST_i} = l_{ST_i} / \left(2R_{C_i}\right)$
$x_{S_i} =$	Tangent distance of any point on spiral with reference to **TS**$_i$ and initial tangent; $x_{S_i} = l_{S_i} \times \left[1 - \frac{\theta_{S_i}^{\ 2}}{10} + \frac{\theta_{S_i}^{\ 4}}{216} + \frac{\theta_{S_i}^{\ 6}}{9,360} + \frac{\theta_{S_i}^{\ 8}}{685,440}\right]$
$x_{ST_i} =$	Total tangent distance from **TS**$_i$ to **SC**$_i$ with reference to initial tangent
$y_{S_i} =$	Tangent offset of any point on spiral with reference to **TS**$_i$ and initial tangent; $y_{S_i} = l_{S_i} \times \left[\frac{\theta_{S_i}}{3} - \frac{\theta_{S_i}^{\ 3}}{42} + \frac{\theta_{S_i}^{\ 5}}{1,320} - \frac{\theta_{S_i}^{\ 7}}{75,600} + \frac{\theta_{S_i}^{\ 9}}{6,894,720}\right]$
$y_{ST_i} =$	Total tangent offset at **SC**$_i$ with reference to **TS**$_i$ and initial tangent;

p_{S_i} =	Offset from the initial tangent to the *PC* of the shifted circle (see Figure 5.5); $p_{S_i} = y_{ST_i} - R_{C_i} \times \left[1 - \cos\left(\theta_{ST_i} \times \frac{180}{\pi}\right)\right]$
k_{S_i} =	Abscissa of the shifted *PC* referred to **TS**$_i$; $k_{S_i} = x_{ST_i} - R_{C_i} \times \sin\left(\theta_{ST_i} \times \frac{180}{\pi}\right)$
L_{TS_i} =	Tangent distance from **TS**$_i$ to **PI**$_i$

Note: equations defining the above notation are based on Hickerson (1964)

In Figure 5.3, the curve *AB* is a spiral transition connecting the tangent $\overline{\mathbf{PI}_{i-1}, \mathbf{PI}_i}$ with point *B* (**SC**$_i$) on a circle. As in the curve *AB*, the transition curve *CD* also adjoins to the tangent $\overline{\mathbf{PI}_i, \mathbf{PI}_{i+1}}$ at point *D* (**ST**$_i$) connecting point *C* (**CS**$_i$) on the same circle. Since these spiral transition curves are symmetric around the common circular curve *BC*, we only describe the geometric specification of the first transition curve.

At point *A* on tangent $\overline{\mathbf{PI}_{i-1}, \mathbf{PI}_i}$ where the spiral curve begins (i.e, at **TS**$_i$), its radius is infinite (i.e., R_{Si}=∞ at **TS**$_i$) and the degree of curvature is zero. The spiral radius gradually decrease along the curve as the spiral distance (l_{Si}) increases from the **TS**$_i$. At point *B* where the spiral curve ends (i.e, at **SC**$_i$), its radius becomes R_{Ci} (i.e., $R_{Si} = R_{Ci}$ at **SC**$_i$). There is an inverse relation between R_{Ci} and l_{Si}. A mathematical relationship between l_{Si}, R_{Si}, and θ_{Si} is summarized in Table 5.2. Note that any point on the spiral curve can be defined with the distance x_{S_i} and offset y_{S_i} with reference to **TS**$_i$. Such a relation is also presented in Table 5.2, and a detailed mathematical description of the relation can be found in Hickerson (1964).

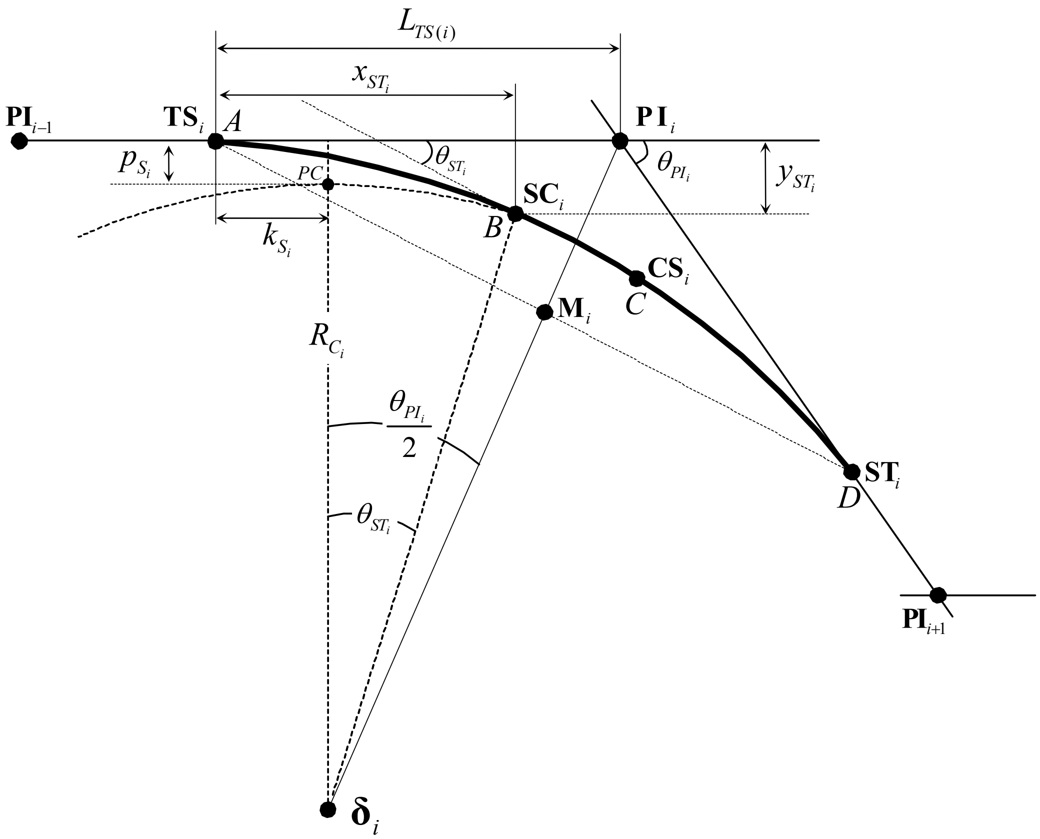


Figure 5.3 Geometric Specification of Horizontal Curved Section with Two Spiral-Transitions

Without violating the design standard (AASHTO, 2001) the minimum radius (denoted as R_m) can be used to fit the radius of a circular curve, and minimum superelevation run-off length (denoted as l_{sr}) can be used to represent the minimum of l_{ST_i} (spiral arc length from $\mathbf{TS}_i$ to $\mathbf{SC}_i$). According to AASHTO (2001), R_m and l_{sr} with given design standards can be expressed as:

$$R_m = \frac{V_d^2}{15(0.01e_s + f_s)} \tag{5.1}$$

$$l_{sr} = \frac{(wn_1)e_s}{\Delta_{MRG}}(b_w) \tag{5.2}$$

where: V_d = Design speed (mph)

e_s = Superelevation rate (%)

f_s = Coefficient of side friction (decimal)

Δ_{MRG} = Maximum relative gradient (%)

n_1 = Number of lanes roated

b_w = Adjustment factor for number of lanes rotated

w = Width of one traffic lane (ft); typically 12 ft

The maximum relative gradient (Δ_{MRG}) is varied with the design speed (V_d) to provide longer runoff lengths at higher speeds (AASHTO, 2001). Values of Δ_{MRG} with respect to different design speeds and those of other design variables defined in equation (5.2) are provided in tabular forms in AASHTO (2001). This equation can be used directly for undivided highways where the cross section is rotated about the highway centerline.

Note that the spiral length can also be specified by the relation, $A^2 = R_{S_i} l_{S_i}$ where A is a constant parameter with range of $2/3R_{S_i} \le A \le 3/2R_{S_i}$ in engineering practices (AASHTO, 2001; Wang et al., 2001). If we adopt this equation, the minimum of l_{ST_i} (denoted as l_{ms}) can be defined as $l_{ms} = 4/9R_m$, which is much simpler than use of l_{sr}. Taking into account all these considerations, the following procedure describes how all the reference points that form the horizontal alignment shown in Figure 5.3 can be found:

Horizontal Alignment Generation Procedure (5.1)

STEP 1: Generate $\mathbf{PI}_i$ of a new alignment along the orthogonal cutting planes ($\forall\, i$ = 1, …, n) using the GAs (Jong, 1998) and the FG and P&R approaches, given its start and end points ($\mathbf{EP}_1$ and $\mathbf{EP}_2$) obtained from the *Endpoint Determination Procedure (5.2)*

STEP 2: Find deflection (intersection) Angle, θ_{PI_i} at $\mathbf{PI}_i$ ($\forall\, i$ = 1, …, n)

$$\theta_{PI_i} = \cos^{-1}\left[\frac{\left(\mathbf{PI}_i - \mathbf{PI}_{i-1}\right)\cdot\left(\mathbf{PI}_{i+1} - \mathbf{PI}_i\right)}{\left\|\mathbf{PI}_i - \mathbf{PI}_{i-1}\right\|\left\|\mathbf{PI}_{i+1} - \mathbf{PI}_i\right\|}\right] \tag{5.3}$$

where: $\| \ \|$ = Length of a vector

$\cdot$ = Dot (inner) product

STEP 3: Find $\mathbf{TS}_i$ and $\mathbf{ST}_i$ ($\forall\, i$ = 1, …, n)

$$\mathbf{TS}_i = \mathbf{PI}_i + L_{TS(i)} \times \frac{\left(\mathbf{PI}_{i-1} - \mathbf{PI}_i\right)}{\left\|\mathbf{PI}_{i-1} - \mathbf{PI}_i\right\|} \tag{5.4}$$

$$\mathbf{ST}_i = \mathbf{PI}_i + L_{TS(i)} \times \frac{\left(\mathbf{PI}_{i+1} - \mathbf{PI}_i\right)}{\left\|\mathbf{PI}_{i+1} - \mathbf{PI}_i\right\|} \tag{5.5}$$

where: $L_{TS(i)}$ = Tangent distance from $\mathbf{TS}_i$ to $\mathbf{PI}_i$

$$= k_{S_i} + \left(R_{C_i} + p_{S_i}\right) \times \tan\left(\frac{\theta_{PI_i}}{2}\right) \tag{5.6}$$

STEP 4: Find $\mathbf{M}_i$ and $\boldsymbol{\delta}_i$ ($\forall\, i$ = 1, …, n)

$$\mathbf{M}_i = \frac{1}{2} \times \left[\mathbf{TS}_i + \mathbf{ST}_i\right] \tag{5.7}$$

$$\boldsymbol{\delta}_i = \mathbf{PI}_i + \left[\left(R_{C_i} + p_{S_i}\right) \times \sec\left(\theta_{PI_i}/2\right)\right] \times \frac{\left(\mathbf{M}_i - \mathbf{PI}_i\right)}{\left\|\mathbf{M}_i - \mathbf{PI}_i\right\|} \tag{5.8}$$

STEP 5: Find $\mathbf{SC}_i$ and $\mathbf{CS}_i$, ($\forall\, i = 1, \ldots, n$)

$$\mathbf{SC}_i = R_{C_i} \times \frac{\left(\mathbf{M}_i - \boldsymbol{\delta}_i\right)}{\left\|\mathbf{M}_i - \boldsymbol{\delta}_i\right\|} \times \mathbf{R}\left(\frac{\theta_{PI_i}}{2} - \theta_{ST_i}\right) \tag{5.9}$$

$$\mathbf{CS}_i = R_{C_i} \times \frac{\left(\mathbf{M}_i - \boldsymbol{\delta}_i\right)}{\left\|\mathbf{M}_i - \boldsymbol{\delta}_i\right\|} \times \mathbf{R}\left(-\frac{\theta_{PI_i}}{2} + \theta_{ST_i}\right) \tag{5.10}$$

where: $\mathbf{R}(\theta)$ = Rotation Matrix

$$= \begin{bmatrix} \cos(\theta) & \text{-}\sin(\theta) \\ \sin(\theta) & \cos(\theta) \end{bmatrix} \tag{5.11}$$

STEP 6: Connect $\mathbf{ST}_i$, $\mathbf{TS}_i$, $\mathbf{SC}_i$, and $\mathbf{CS}_i$ ($\forall\, i = 1, \ldots, n$) consecutively

Representation of the vertical alignment, which consists of grade tangents connected with parabolic vertical curves, may be found in Jong (1998).

5.2 Modeling Highway Endpoints

As stated previously, in the earlier version of the highway alignment optimization (HAO) model, the start and end points of a new highway are assumed to be predetermined by model users before the optimization process. However, such a strong assumption is relaxed in this dissertation. Here, the highway endpoints are also considered as decision variables in the model rather than given inputs. Note that additional information on the existing road alignments (i.e., horizontal and vertical profiles) may be required for this relaxation.

Here, we make the reasonable assumption that the model users can specify several preferred sub-segments along the existing roads. Such an assumption is realistic since there may be many critical points along the existing roads which are not suitable as junction points between the new and existing roads. For instance, near interchanges (or intersections), sharply curved sections in which drivers' sight-distances are insufficient, and bridge sections on existing roads may be unsuitable as junction points. These critical points should be prescreened before the optimization process. Given the basic information of the existing road identified from a GIS database (e.g., a horizontal profile of the existing road and its corresponding elevation data) and user preferences (i.e., preferred road segments) for the endpoints of a new highway, a method for determining the highway endpoints is described below.

5.2.1 Determination of Highway Endpoints

It is assumed that a piecewise linear data format is used to save and extract the coordinates of an existing road. This makes it easy to represent the existing road in the model with a simple vector operation. A sufficient number of points may be required for representing the road realistically.

Suppose that ten intermediate points are successively specified by the model users along the existing road to which a new highway will be connected (see Figure 5.4(a)). Then, piecewise linear segments obtained from the connection of the intermediate points roughly outline the existing road. The XY coordinates of all the intermediate points can be easily obtained from an input GIS database for the study area, and ground elevations of those points may also be directly obtained through a DEM[10].

It should be noted here that we do not need XY coordinates for all the intermediate points (here ten) to generate the highway endpoints. If two sub-segments of the existing highway are selected for domains of the possible highway end point (or start point), as shown in Figure 5.4, only coordinates of the four intermediate points (here $\mathbf{I}_{E1}$, $\mathbf{I}_{E2}$, $\mathbf{I}_{E3}$, and $\mathbf{I}_{E4}$) must be identified. Possible locations of the endpoint are continuous along the specified road segments and will be generated with the endpoint determination procedure below.

---

[10] The digital elevation model (DEM) is the most common basis used in many GISs as a type of digital terrain model (DTM), recording a topographical representation of the terrain of the Earth or another surface in digital format. The DEM normally divides the area into rectangular pixels and stores the elevation of each pixel.

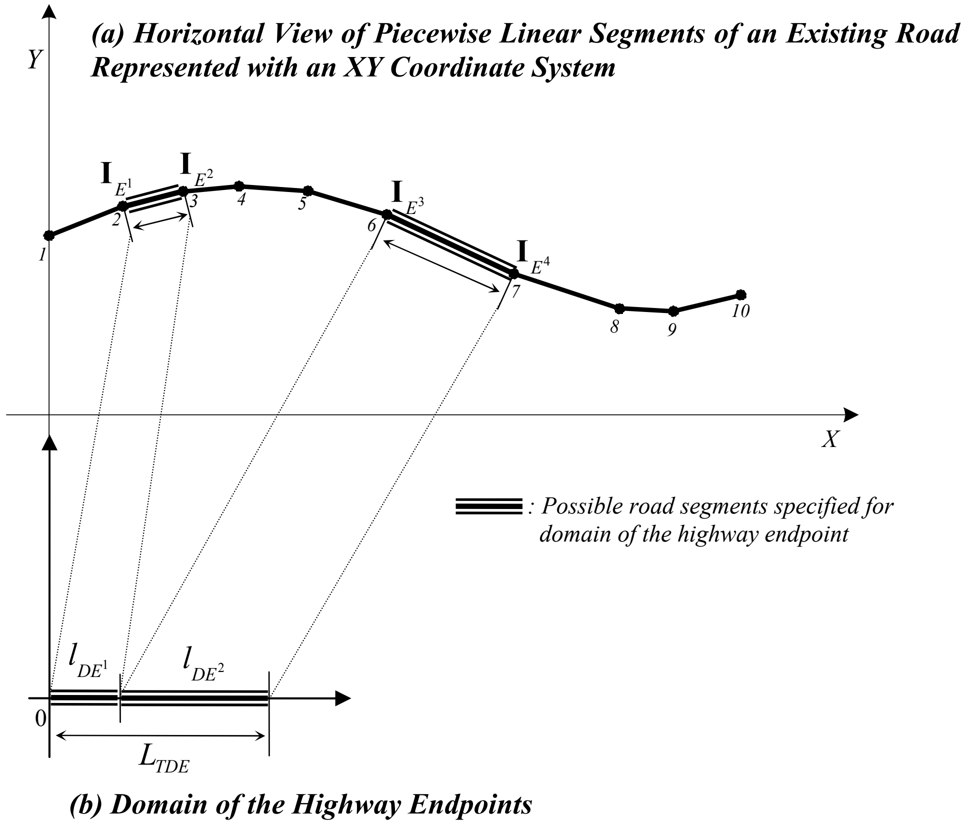


Figure 5.4 Representation of Domain of the Possible Endpoints

Let the length of each road segment specified be l_{DE}. Then, the domain of the possible endpoints can reduce to $L_{TDE} = \sum_{j=1}^{n_{seg}} l_{DE^j}$, where n_{seg} is total number of road segments specified. Given the ground elevation database (DEM) and XY coordinates of all the specified intermediate points (denoted as $\mathbf{I}_{E^i} = (x_{I_{E^i}}, y_{I_{E^i}})\ \forall i$), an algorithm for determining possible locations of the highway endpoint is developed below. Note that three additional reference points, which are necessary for representing the three-leg structures of the endpoint, are also found from the following procedure. The notation used in the procedure is summarized in Table 5.3.

Endpoint Determination Procedure (5.2)

STEP 1: For all pairs of the intermediate points specified, calculate l_{DE^j} and L_{TDE}

for $i = 1$ to $n_i - 1$

$$l_{DE^j} = \left\| \mathbf{I}_{E^{i+1}} - \mathbf{I}_{E^i} \right\| = \sqrt{\left(x_{I_{E^{i+1}}} - x_{I_{E^i}}\right)^2 + \left(y_{I_{E^{i+1}}} - y_{I_{E^i}}\right)^2}$$

$i = i + 1$

end

$$L_{TDE} = \sum_{j=1}^{n_{seg}} l_{DE^j}$$

STEP 2: Find l_{Temp} randomly between 0 and L_{TDE}

$$l_{Temp} = r_c\left[0,\ L_{TDE}\right]$$

STEP 3: Find selected segment k and l_{seg}

if $l_{Temp} \le l_{DE^1}$, $\quad \rightarrow\ l_{seg} = l_{Temp}$, $k = 1$ and go to STEP 4

else

for $i = 2$ to n_{seg}

if $l_{Temp} \le \sum_{j=1}^{i} l_{DE^j}$, $\quad \rightarrow\ l_{seg} = l_{Temp} - \sum_{j=1}^{i-1} l_{DE^j}$, $k = i$ and go to STEP 4

else $\quad \rightarrow\ i = i + 1$

end

end

STEP 4: Find XY coordinates of three reference points ($\mathbf{RP}_0$, $\mathbf{RP}_1$, and $\mathbf{RP}_3$) required for modeling structures of the highway endpoints

$$\begin{bmatrix} x_{RP_0} \\ y_{RP_0} \end{bmatrix} = \mathbf{I}_{E^{2k-1}} + l_{seg} \times \frac{\mathbf{I}_{E^{2k}} - \mathbf{I}_{E^{2k-1}}}{\left\| \mathbf{I}_{E^{2k}} - \mathbf{I}_{E^{2k-1}} \right\|}$$

$$\begin{bmatrix} x_{RP_1} \\ y_{RP_1} \end{bmatrix} = \mathbf{I}_{E^{2k-1}} + \left(l_{seg} - \Delta l_{seg}\right) \times \frac{\mathbf{I}_{E^{2k}} - \mathbf{I}_{E^{2k-1}}}{\left\| \mathbf{I}_{E^{2k}} - \mathbf{I}_{E^{2k-1}} \right\|}$$

$$\begin{bmatrix} x_{RP_2} \\ y_{RP_2} \end{bmatrix} = \mathbf{I}_{E^{2k-1}} + \left(l_{seg} + \Delta l_{seg}\right) \times \frac{\mathbf{I}_{E^{2k}} - \mathbf{I}_{E^{2k-1}}}{\left\| \mathbf{I}_{E^{2k}} - \mathbf{I}_{E^{2k-1}} \right\|}$$

STE P 5 : Find Z coordinates of the three reference points

$\rightarrow$ Compute ground elevations of the three reference points ($z_{RP_0}, z_{RP_1}, z_{RP_2}$) using the planar-interpolation method (Kim, 2001) given with the GIS elevation database (DEM)

STE P 6 : Find a three-dimensional (3D) highway endpoint (either start or end point)

$$\mathbf{EP} = \mathbf{RP}_0 = \begin{bmatrix} x_{RP_0} \\ y_{RP_0} \\ z_{RP_0} \end{bmatrix}$$ Case 1: for at-grade intersected structures

$$\mathbf{EP} = \begin{bmatrix} x_{RP_0} \\ y_{RP_0} \\ z_{RP_0} + H_m \end{bmatrix}$$ Case 2: for grade-separated structures with overpassing existing highways

$$\mathbf{EP} = \begin{bmatrix} x_{RP_0} \\ y_{RP_0} \\ z_{RP_0} - H_m \end{bmatrix}$$ Case 3: for grade-separated structures with underpassing existing highways

Table 5.3 Notation Used for Endpoint Determination Procedure

Notation	Descriptions
$\mathbf{I}_{E^i}, \mathbf{I}_{E^{i+1}}$ =	A pair of intermediate points specified for representing a preferred road segment of the highway endpoint along the existing road, for i=1, …, n_i
n_i=	Total number of intermediate points specified
l_{DE^j} =	Length of each road segments specified, for j=1, …, n_{seg}
n_{seg}=	Total number of the road segments specified; $n_{seg} = n_i/2$
L_{TDE}=	Total length of the road segments specified
$r_c[A, B]$ =	A random value from a continuous uniform distribution whose domain is within the interval [A, B]
l_{Temp}=	A provisional random value from $r_c[A, B]$
k=	The road segment selected for the highway endpoint from the random search process
$\mathbf{I}_{E^{2k-1}}, \mathbf{I}_{E^{2k}}$ =	A pair of intermediate points corresponding to the selected road segment k for the highway endpoint
l_{Seg}=	Distance from $\mathbf{I}_{E^{2k-1}}$ to the highway endpoint

Δl_{seg} =	A provisional distance used for finding the reference points $\mathbf{RP}_1$ and $\mathbf{RP}_2$; (typically less than 10 ft)
H_m=	Minimum vertical clearance for grade separation
EP=	A 3D point found for the highway endpoint (either start or end point); $\mathbf{EP} = (x_{EP}, y_{EP}, z_{EP})$
$\mathbf{RP}_0, \mathbf{RP}_1, \mathbf{RP}_2$=	3D reference points required to model three-leg structures of the highway endpoint (refer to Figures 5.6 or 5.7); $\mathbf{RP}_0=\left(x_{RP_0}, y_{RP_0}, z_{RP_0}\right), \mathbf{RP}_1=\left(x_{RP_1}, y_{RP_1}, z_{RP_1}\right), \mathbf{RP}_2=\left(x_{RP_2}, y_{RP_2}, z_{RP_2}\right)$

In Step 1, the length of each road segment defined by the model users is calculated, and then a temporary random value is generated from the uniform random process with domain 0 to L_{TDE} through Step 2. From Step 3 to Step 4, XY coordinates of the three reference points are computed with a simple vector operation. In Step 5, elevations (Z coordinates) of the reference points are obtained through the planar interpolation method (Kim, 2001), given with XY coordinates of those points and the input GIS elevation database (DEM). Note that we may obtain the elevations directly from the provided input DEM without the interpolation method; however, this is less desirable since they may not be sufficiently accurate to use directly in the alignment design process.

Finally, XYZ coordinates of the highway endpoint **EP** (either start or end point of the new alignment) are computed based on those of the reference point $\mathbf{RP}_0$ found in Steps 4 and 5. Elevation (Z_{EP}) of **EP** can be varied depending on type of structures considered for representing the endpoint. If an at-grade intersection (e.g., 3-leg intersection or roundabout) is considered, the XYZ coordinates of **EP** become those of $\mathbf{RP}_0$. However, if a grade separated structure (e.g., a trumpet interchange) is considered, the XY coordinates of **EP** become those of $\mathbf{RP}_0$, while its ground elevation is either elevated to $Z_{RP0}+H_m$ (for over-passing the existing road) or lowered to $Z_{RP0}-H_m$ (for under-passing the existing road). Note that type of structures used for representing the endpoint and their minimum vertical clearance (H_m) can be specified by the model users.

GA Operators for Endpoint Generation

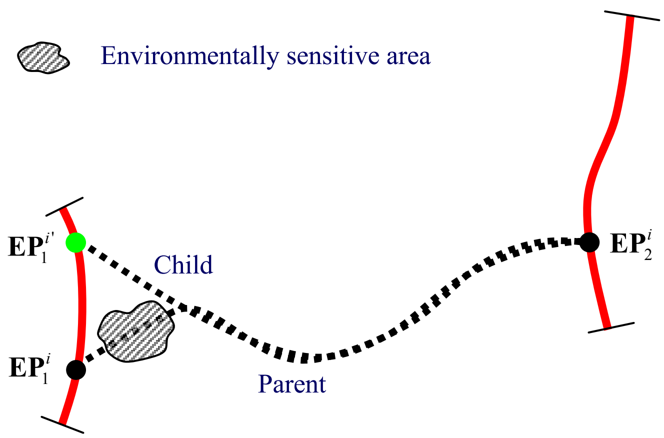
Three customized GA operators are employed for evolving the highway endpoints during the alignment search process. These are:

- Uniform mutation
- One-point crossover
- Two-point crossover

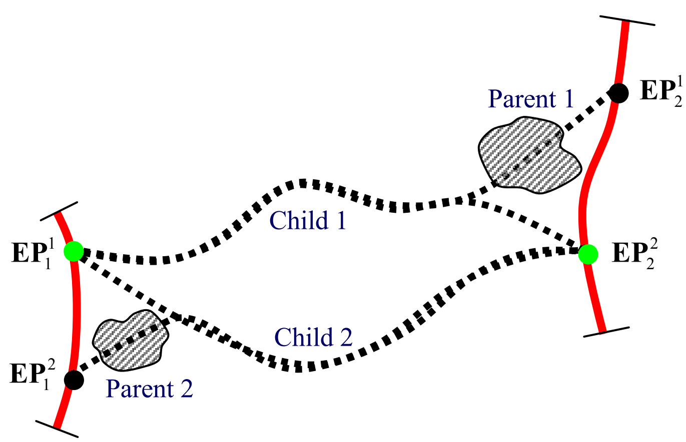
The uniform mutation operator is used for arbitrarily altering the endpoints (either start or end points) of selected chromosomes[11]. Let the chromosome to be mutated be $\Lambda=\left[\mathbf{EP}_1,\ \mathbf{PI}_1,......,\ \mathbf{PI}_n,\ \mathbf{EP}_2\right]$ and either $\mathbf{EP}_1$ or $\mathbf{EP}_2$ be selected to apply the uniform mutation. Then, the EP will be replaced with a new endpoint from the random search process of the *Endpoint Determination Procedure*. Figure 5.5 (a) shows a good example when this mutation operator works well during the optimization process. As shown in the figure, a good offspring is generated by

[11] The highway alignments are represented with chromosomes in the model (see section 5.2.1); readers may refer to Jong (1998) and Jong and Schonfeld (2003) for a detailed description of the GA encoding.

replacing the start point ($\mathbf{EP}_1^i$) of a selected parent alignment with new one ($\mathbf{EP}_1^{i+1}$), while inheriting the other genes (i.e., a set of PI's and $\mathbf{EP}_2^i$) from the parent. The resulting offspring completely avoids a no-go area (e.g., environmentally sensitive area) in the search space after the endpoint mutation.



(a) An Example of a Uniform Mutation Operator



(a) An Example of a One-point Crossover Operator

Figure 5.5 Examples of Endpoint Operators

Besides the mutation operators, two crossover operators (one-point and two-point) are used for endpoints evolution. These operators allow the new offspring to inherit good genes from the parents by swapping their endpoints. The concept of the one-point crossover is to exchange only one endpoint (either the start or end point) between two selected parents, while two-point crossover swaps both endpoints of the two parents simultaneously. Suppose that two parents (Λ^1 and Λ^2) are selected for the crossover operation, where Λ^1 = $\left[\mathbf{EP}_1^1,\ \mathbf{PI}_1^1, \ldots\ldots,\ \mathbf{PI}_n^1,\ \mathbf{EP}_2^1\right]$ and Λ^2 = $\left[\mathbf{EP}_1^2,\ \mathbf{PI}_1^2, \ldots\ldots,\ \mathbf{PI}_n^2,\ \mathbf{EP}_2^2\right]$. Then, the resulting offspring from the one-point crossover operator are:

$$\Lambda^{1'} = \left[\mathbf{EP}_1^1,\ \mathbf{PI}_1^1, \ldots\ldots,\ \mathbf{PI}_n^1,\ \mathbf{EP}_2^2\right] \qquad \Lambda^{2'} = \left[\mathbf{EP}_1^1,\ \mathbf{PI}_1^2, \ldots\ldots,\ \mathbf{PI}_n^2,\ \mathbf{EP}_2^2\right]$$

$$\text{or} \qquad\qquad\qquad\qquad \text{or}$$

$$= \left[\mathbf{EP}_1^2,\ \mathbf{PI}_1^1, \ldots\ldots,\ \mathbf{PI}_n^1,\ \mathbf{EP}_2^1\right] \qquad \left[\mathbf{EP}_1^2,\ \mathbf{PI}_1^2, \ldots\ldots,\ \mathbf{PI}_n^2,\ \mathbf{EP}_2^1\right]$$

The resulting offspring from two-point crossover are:

$$\Lambda^{1'} = \left[\mathbf{EP}_1^2,\ \mathbf{PI}_1^1, \ldots\ldots,\ \mathbf{PI}_n^1,\ \mathbf{EP}_2^2\right]$$

$$\Lambda^{2'} = \left[\mathbf{EP}_1^1,\ \mathbf{PI}_1^2, \ldots\ldots,\ \mathbf{PI}_n^2,\ \mathbf{EP}_2^1\right]$$

Figure 5.5 (b) shows a successful example of the one-point crossover operator. Child 1 ($\Lambda^{1'}$) inherits its end point from parent 2 (i.e., $\mathbf{EP}_2^2$), while the other genes are inherited from parent 1 (Λ^1). Similarly, child 2 ($\Lambda^{2'}$) inherits its start point from parent 1 (i.e., $\mathbf{EP}_1^1$), while the other genes are taken over from parent 2 (Λ^2).

More detailed genetic encoding of the GA operators employed for the endpoint generation may be found in Jong (1998).

5.2.2 Representation of Highway Endpoints

A method for finding the endpoint of the new alignment (along the existing road) is presented in the previous section. This section tries to realistically represent the endpoint with some 3-leg structures which are most commonly used in highway design process for the highway cross-points, and their cost functions are also proposed here. Many types of 3-leg structures are considered where a new highway diverges from an existing road. Among them, trumpet (or T-type) interchanges, at-grade intersections, and roundabouts are modeled here. Other complex and large interchanges are not considered here since they require their own vast research areas. Cost functions for some simple 4-leg structures (e.g., 4-leg intersections, clover and diamond interchanges) may be found in Kim (2001).

5.2.2.1 Three-Leg Intersections

Intersection pavement cost, right-of-way cost, and earthwork cost are major construction cost components of a 3-leg intersection. These cost items can be approximately estimated with a centerline drawing of roadways associated with the intersection. As shown in Figure 5.6, we basically assume that the 3-leg intersection considered in the model has two separated small right-turn roads (ramps). The right turn ramps can be neglected if necessary; however, they may be used for a 3-leg intersection in a rural area.

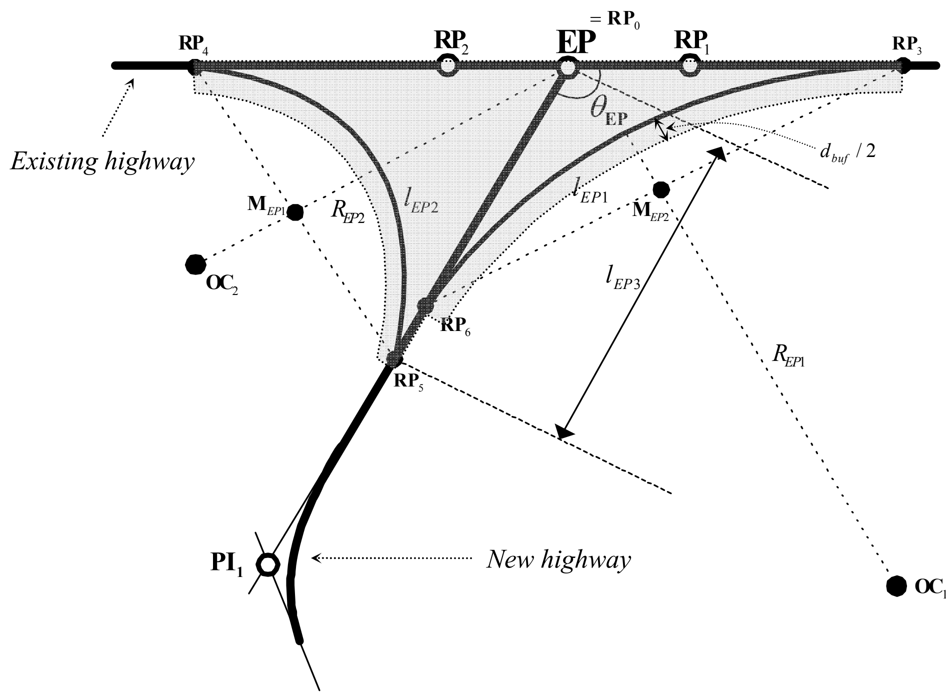


Figure 5.6 Centerline Drawing of a Three-Leg Intersection with a Set of Reference Points

Finding Reference Points for Representing 3-Leg Intersections

At least four important reference points are needed to describe a 3-leg intersection structure. These are the three reference points (**EP**, $\mathbf{RP}_1$, and $\mathbf{RP}_2$) found from the *Endpoint Determination Procedure* in Section 5.2.1, and the first or last PI of the new highway alignment (denoted as $\mathbf{PI}_1$ in Figure 5.6). These reference points are used to calculate the cross-angle, denoted as θ_{EP}, between the new and existing highways, and to find additional reference points ($\mathbf{RP}_3$, $\mathbf{RP}_4$, $\mathbf{RP}_5$, $\mathbf{RP}_6$, $\mathbf{OC}_1$, $\mathbf{OC}_2$) needed to outline the intersection structure, as shown in Figure 5.6. The coordinates of the additional reference points can be found simply by using a vector operation, and some design standards (e.g., radii of the two right-turn roads) associated with the intersection structure are needed for the operation. The following procedure illustrates how the cross-angle (θ_{EP}) and the coordinates of all the reference points are found.

STEP 1: Find $\mathbf{RP}_3$, $\mathbf{RP}_4$, $\mathbf{RP}_5$, and $\mathbf{RP}_6$

$$\mathbf{RP}_3 = \mathbf{EP} + R_{EP1} \tan\left(\frac{\theta_{EP}}{2}\right)\frac{\mathbf{RP}_1 - \mathbf{EP}}{\|\mathbf{RP}_1 - \mathbf{EP}\|} \tag{5.12}$$

$$\mathbf{RP}_4 = \mathbf{EP} + R_{EP2} \tan\left(\frac{\pi - \theta_{EP}}{2}\right)\frac{\mathbf{RP}_2 - \mathbf{EP}}{\|\mathbf{RP}_2 - \mathbf{EP}\|} \tag{5.13}$$

$$\mathbf{RP}_5 = \mathbf{EP} + R_{EP2} \tan\left(\frac{\pi - \theta_{EP}}{2}\right)\frac{\mathbf{PI}_1 - \mathbf{EP}}{\|\mathbf{PI}_1 - \mathbf{EP}\|} \tag{5.14}$$

$$\mathbf{RP}_6 = \mathbf{EP} + R_{EP1} \tan\left(\frac{\theta_{EP}}{2}\right)\frac{\mathbf{PI}_1 - \mathbf{EP}}{\|\mathbf{PI}_1 - \mathbf{EP}\|} \tag{5.15}$$

where: θ_{EP} = Highway cross-angle between the new and existing roads;

$\pi/3 \le \theta_{EP} \le 2\pi/3$ (from AASHTO)

R_{EP1} = Radius of the right turn road 1

R_{EP2} = Radius of the right turn road 2

$\| \;\|$ = Length of a vector and,

$$\theta_{EP} = \cos^{-1}\left[\frac{(\mathbf{EP} - \mathbf{RP}_1)\cdot(\mathbf{PI}_1 - \mathbf{EP})}{\|\mathbf{EP} - \mathbf{RP}_1\|\|\mathbf{PI}_1 - \mathbf{EP}\|}\right] \tag{5.16}$$

where: $\cdot$ = Inner(dot) product

STEP 2: Find $\mathbf{M}_{EP1}$, $\mathbf{M}_{EP2}$, $\mathbf{OC}_1$, and $\mathbf{OC}_2$

$$\mathbf{M}_{EP1} = \mathbf{RP}_6 + R_{EP1} \sin\left(\frac{\pi - \theta_{EP}}{2}\right)\frac{\mathbf{RP}_3 - \mathbf{RP}_6}{\|\mathbf{RP}_3 - \mathbf{RP}_6\|} \tag{5.17}$$

$$\mathbf{OC}_1 = \mathbf{EP} + \frac{R_{EP1}}{\cos((\pi - \theta_{EP})/2)}\frac{\mathbf{M}_{EP1} - \mathbf{EP}}{\|\mathbf{M}_{EP1} - \mathbf{EP}\|} \tag{5.18}$$

$$\mathbf{M}_{EP2} = \mathbf{RP}_5 + R_{EP2} \sin\left(\frac{\theta_{EP}}{2}\right)\frac{\mathbf{RP}_4 - \mathbf{RP}_5}{\|\mathbf{RP}_4 - \mathbf{RP}_5\|} \tag{5.19}$$

$$\mathbf{OC}_2 = \mathbf{EP} + \frac{R_{EP2}}{\cos(\theta_{EP}/2)}\frac{\mathbf{M}_{EP2} - \mathbf{EP}}{\|\mathbf{M}_{EP2} - \mathbf{EP}\|} \tag{5.20}$$

Note that in the model, the range of the highway cross-angle θ_{EP} is restricted to $\pi/3 \le \theta_{EP} \le 2\pi/3$. Such a restriction follows the AASHTO (2001) standard prohibiting a sharp intersection cross-angle below $\pi/3$.

Cost Functions for 3-Leg Intersections

We now develop the major intersection cost functions based on the all the reference points found in the above and some input parameters (e.g., road widths and several unit costs) associated with the intersection design. First, the intersection pavement cost can be roughly estimated with centerline distances and widths of all approach roads in the intersection and unit pavement cost:

$$\begin{aligned} C_{IS_P} &= \left[\left(l_{EP1} \times W_1 \right) + \left(l_{EP2} \times W_2 \right) + \left(l_{EP3} \times W_3 \right) \right] \times K_p \\ &= \left[\begin{array}{l} \left(R_{EP1} \times \theta_{EP} \times W_1 \right) + \left(R_{EP2} \times \left(\pi - \theta_{EP} \right) \times W_2 \right) + \\ \max \left\{ R_{EP1} \times \tan\left(\dfrac{\theta_{EP}}{2} \right),\ R_{EP2} \times \tan\left(\dfrac{\pi - \theta_{EP}}{2} \right) \right\} \times W_3 \end{array} \right] \times K_p \end{aligned} \tag{5.21}$$

where: C_{IS_P} = Intersection pavement cost

K_p = Unit pavement cost (\$/ft^2)

l_{EP1} = Arc length (ft) of the right-turn road approaching from the new highway to the existing highway

l_{EP2} = Arc length (ft) of the right-turn road approaching from the existing road to the new highway

l_{EP3} = Length of the longer tangent obtained from between the two right-turn roads

W_1, W_2, and W_3 = Widths of road segments corresponding to l_{EP1}, l_{EP2}, and l_{EP3}, respectively

Another intersection cost component that must be considered is the intersection right-of-way (ROW) cost. As shown in Figure 5.6, the intersection ROW area (shaded in the figure) can be described with several reference points found from the steps in the above. The ROW area can be estimated as follows:

$$\begin{aligned} A_{IS_R} = &\left\{ \left[{R_{EP1}}^2 \times \tan\left(\frac{\pi - \theta_{EP}}{2} \right) \right] - \left[\left(R_{EP1} - \left(d_{buf} / 2 \right) \right)^2 \times \left(\pi - \theta_{EP} \right) \right] \right\} \\ &+ \left\{ \left[{R_{EP2}}^2 \times \tan\left(\frac{\theta_{EP}}{2} \right) \right] - \left[\left(R_{EP2} - \left(d_{buf} / 2 \right) \right)^2 \times \theta_{EP} \right] \right\} \end{aligned} \tag{5.22}$$

where: A_{IS_R} = Intersection right-of-way area (sq.ft) required
d_{buf} = Buffer width (ft) required for intersection construction as shown in Figure 5.6; $d_{buf} \geq W_i \ \forall i$

Given the intersection ROW area found, a method is needed to estimate the intersection ROW cost by identifying properties affected by the new intersection structure. To do this, Jha's (2000) method, which is incorporated in equations (7.4) of Section 7.1, is employed.

For computing the earthwork cost of the 3-leg intersections, the earthwork volume required should be estimated first. The input ground elevation databases (DEM) and coordinates of all the reference points found in the previous section are used for estimating the intersection earthwork volume. Kim (2001) proposed a method for estimating the earthwork volume of 4-leg intersections. His method is also applicable to 3-leg intersections. In Kim (2001), the total intersection earthwork volume (E_V) can be expressed as:

$$E_V = \sum_{i=1}^{n_p} A_i^b \times \left(Z_{b_i}^{ave} - Z_{g_i}^{ave} \right) \tag{5.23}$$

where: E_V = Estimated earthwork volume

n_p = Total number of parcels representing the intersection

A_i^b = Base area of parcel i

$Z_{b_i}^{ave}$ = Average base-elevation of parcel i

$Z_{b_i}^{ave}$ = Average ground elevation of parcel i

Given the estimated earthwork volume found from equation (5.23), the intersection earthwork cost (C_{IS_E}) can be expressed as:

$$C_{IS_E} = E_V K_f \tag{5.24}$$

where: C_{IS_E} = Earthwork cost for the 3-leg intersection

K_f = Unit fill cost ($/ft^3)

Kim (2001) provides additional details for equations (5.23) and (5.24).

5.2.2.2 Trumpet Interchanges

A typical (simple) trumpet interchange is modeled in this section. Figure 5.7 shows the centerline drawing of the structure. It comprises a small bridge for grade separation and several ramps.

As in the 3-leg intersections, the construction cost of the trumpet interchange can be subdivided into several sub-categories. These are (1) interchange pavement cost, (2) interchange ROW cost, (3) small bridge cost for grade separation, and (4) interchange earthwork cost.

$Case1$: A trumpet interchange skewed rightward ($\pi / 2 < \theta_{EP}$)

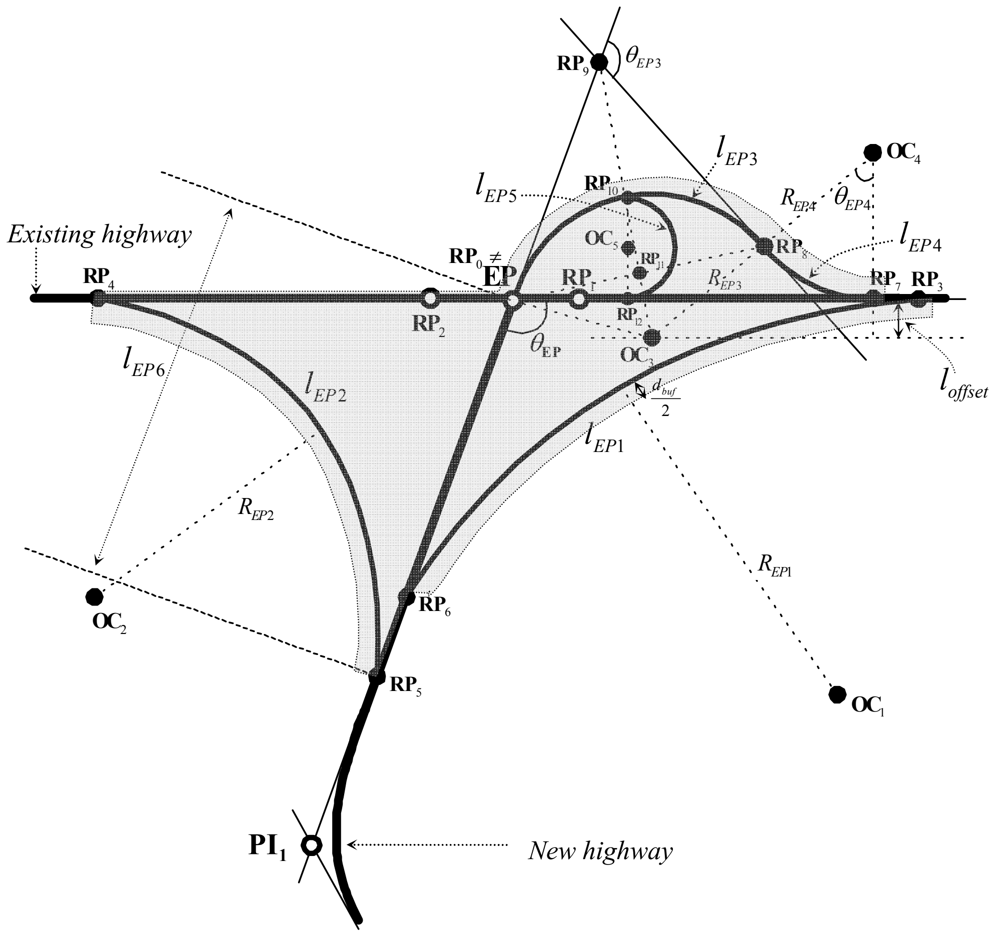


Figure 5.7 Centerline Drawing of a Typical Trumpet Interchange with a Set of Reference Points

Finding Reference Points for Representing Trumpet Interchanges

Two different cases of the trumpet interchange can be considered in modeling it. In the first case, shown in Figure 5.7, the intersection angle (θ_{EP}) of the two highways (new and existing roads) exceeds 90 degrees (i.e., $\pi / 2 < \theta_{EP}$). In the other case, θ_{EP} is less than and equal to 90 degrees (i.e., $\pi / 2 \geq \theta_{EP}$). As shown in Figure 5.7, the outside ramps (of which lengths are denoted as l_{EP3}, l_{EP4}, and l_{EP5} in the figure) are placed on the right side of the new alignment if $\pi/2 < \theta_{EP}$ (Case 1). However, those ramps would be located on the left side of the new alignment if $\pi/2 \geq \theta_{EP}$ (Case 2). Since the same modeling procedure can be applied to both cases of the trumpet interchange, this dissertation describes the former case only, although the model can actually deal with both.

Note that the geometric design of the trumpet interchanges is relatively more complex than that of the 3-leg intersection, and thus more reference points must be identified. As in the 3-leg intersection model, the highway-cross angle θ_{EP} and additional reference points ($\mathbf{RP}_3$, $\mathbf{RP}_4$, $\mathbf{RP}_5$, $\mathbf{RB}_6$, $\mathbf{RB}_7$, $\mathbf{RB}_8$, $\mathbf{RP}_9$, $\mathbf{RP}_{10}$, $\mathbf{RP}_{11}$, $\mathbf{RP}_{12}$, $\mathbf{OC}_1$, $\mathbf{OC}_2$ $\mathbf{OC}_3$, $\mathbf{OC}_4$, and $\mathbf{OC}_5$), which are required to outline the interchange structure as shown in Figure 5.7, can be found from a simple vector operation given the four important points ($\mathbf{EP}$, $\mathbf{RP}_1$, $\mathbf{RP}_2$, and $\mathbf{PI}_1$) and with several design inputs associated with its construction. Among all the reference points, some (such as $\mathbf{RP}_3$, $\mathbf{RP}_4$, $\mathbf{RP}_5$, $\mathbf{RB}_6$, $\mathbf{OC}_1$, and $\mathbf{OC}_2$) can be found with equations (5.12) through (5.20) defined in the previous section. The remaining reference points can be found with the following procedure:

STEP 1: Find $\mathbf{RP}_9$

$$\mathbf{RP}_9 = \mathbf{EP} + R_{EP3} \tan\left(\frac{\theta_{EP3}}{2}\right) \frac{\mathbf{EP} - \mathbf{PI}_1}{\left\|\mathbf{EP} - \mathbf{PI}_1\right\|} \tag{5.25}$$

where: θ_{EP3} = Centeral angle of the ramp 3

R_{EP3} = Radius of the ramp 3, and

$$\theta_{EP3} = \pi - \theta_{EP} + \cos^{-1}\left(\frac{R_{EP4} + l_{offset}}{R_{EP3} + R_{EP4}}\right) \tag{5.26}$$

where: R_{EP4} = Radius of the ramp 4

l_{offset} = Perpendicular distance from $\mathbf{OC}_3$ to the existing road

$\mathbf{OC}_3$ = Center point of ramp 3, and

$$l_{offset} = R_{EP3}\sin\left(\theta_{EP} - \frac{\pi}{2}\right) \tag{5.27}$$

STEP 2: Find $\mathbf{OC}_3$

$$\mathbf{OC}_3 = \mathbf{EP} - R_{EP3}\left(\frac{\mathbf{V}_1}{\|\mathbf{V}_1\|}\right) \tag{5.28}$$

where: $\mathbf{V}_1$ = Unit vector orthogonal to $\overrightarrow{\mathbf{PI}_1,\mathbf{EP}}$ at $\mathbf{EP}$, and

$$\mathbf{V}_1 = \mathbf{EP} - \left(\frac{\mathbf{EP}\cdot\overrightarrow{\mathbf{PI}_1,\mathbf{EP}}}{\overrightarrow{\mathbf{PI}_1,\mathbf{EP}}\cdot\overrightarrow{\mathbf{PI}_1,\mathbf{EP}}}\right)\overrightarrow{\mathbf{PI}_1,\mathbf{EP}} \tag{5.29}$$

where: $\overrightarrow{\mathbf{PI}_1,\mathbf{EP}} = \mathbf{EP} - \mathbf{PI}_1$

STEP 3: Find $\mathbf{RP}_{10}$, $\mathbf{RP}_{11}$, $\mathbf{RP}_8$, and $\mathbf{OC}_4$

$$\mathbf{RP}_{10} = \mathbf{OC}_3 + R_{EP3}\frac{\mathbf{RP}_9 - \mathbf{OC}_3}{\|\mathbf{RP}_9 - \mathbf{OC}_3\|} \tag{5.30}$$

$$\mathbf{RP}_{11} = \mathbf{OC}_3 + R_{EP3}\cos\left(\frac{\theta_{EP3}}{2}\right)\frac{\mathbf{RP}_9 - \mathbf{OC}_3}{\|\mathbf{RP}_9 - \mathbf{OC}_3\|} \tag{5.31}$$

$$\mathbf{RP}_8 = \mathbf{EP} + R_{EP3}\sin\left(\frac{\theta_{EP3}}{2}\right)\frac{\mathbf{RP}_{11} - \mathbf{EP}}{\|\mathbf{RP}_{11} - \mathbf{EP}\|} \tag{5.32}$$

$$\mathbf{OC}_4 = \mathbf{OC}_3 + \left(R_{EP3} + R_{EP4}\right)\frac{\mathbf{RP}_8 - \mathbf{OC}_3}{\|\mathbf{RP}_8 - \mathbf{OC}_3\|} \tag{5.33}$$

STEP 4: Find $\mathbf{RP}_{12}$, $\mathbf{RP}_7$, and $\mathbf{OC}_5$

$$\mathbf{RP}_{12} = \mathbf{EP} + \begin{bmatrix} R_{EP3}\cos\left(\theta_{EP} - \frac{\pi}{2}\right) - \\ R_{EP3}\cos\left(\theta_{EP} - \frac{\pi}{2} + \frac{\theta_{EP3}}{2}\right) \end{bmatrix}\frac{\mathbf{RP}_1 - \mathbf{EP}}{\|\mathbf{RP}_1 - \mathbf{EP}\|} \tag{5.34}$$

$$\mathbf{RP}_7 = \mathbf{EP} + \begin{bmatrix} R_{EP3}\cos\left(\theta_{EP} - \dfrac{\pi}{2}\right) + \\ \left(R_{EP3} + R_{EP4}\right)\cos\left(\dfrac{\pi}{2} - \theta_{EP4}\right) \end{bmatrix} \frac{\mathbf{RP}_1 - \mathbf{EP}}{\left\|\mathbf{RP}_1 - \mathbf{EP}\right\|} \tag{5.35}$$

where: θ_{EP4}= Centeral angle of the ramp 4, and

$$\theta_{EP4} = \cos^{-1}\left(\frac{R_{EP4} + l_{offset}}{R_{EP3} + R_{EP4}}\right) \tag{5.36}$$

$$\mathbf{OC}_5 = \mathbf{RP}_{12} + R_{EP5}\frac{\mathbf{RP}_{10} - \mathbf{RP}_{12}}{\left\|\mathbf{RP}_{10} - \mathbf{RP}_{12}\right\|} \tag{5.37}$$

where: R_{EP5} = Radius of the ramp 5, and

$$R_{EP5} = \frac{R_{EP3}\sin\left(\left(\theta_{EP} + \theta_{EP4}\right)/2\right)}{2} \tag{5.38}$$

Cost Functions for Trumpet Interchanges

We now develop cost functions for the trumpet interchange based on all the reference points found above. Several input parameters (e.g., road widths and unit costs associated with the interchange construction) are also required to model the cost functions.

The pavement cost of the trumpet interchange can be roughly estimated with centerline distances and widths of all approach roads to the structure and with unit pavement cost. The trumpet interchange pavement cost can be expressed as:

$$C_{IC_P} = \begin{bmatrix} (l_{EP1}W_1) + (l_{EP2}W_2) + (l_{EP3}W_3) + \\ (l_{EP4}W_4) + (l_{EP5}W_5) + (l_{EP6}W_6) \end{bmatrix} K_p$$

$$= \begin{bmatrix} (R_{EP1}\theta_{EP}W_1) + (R_{EP2}(\pi - \theta_{EP})W_2) + \\ (R_{EP3}\theta_{EP3}W_3) + (R_{EP4}\theta_{EP4}W_4) + (R_{EP5}\pi W_5) \\ + \min\left\{ R_{EP1} \tan\left(\frac{\theta_{EP}}{2}\right),\ R_{EP2} \tan\left(\frac{\pi - \theta_{EP}}{2}\right) \right\} W_6 \end{bmatrix} K_p \tag{5.39}$$

where: C_{IC_P} = Pavement cost of trumpet interchange

l_{EP1}, l_{EP2}, l_{EP3}, l_{EP4}, and l_{EP5} are arc lengths of the trumpet-interchange ramps as shown in Figure 5.7.

W_1, W_2, W_3, W_4, and W_5 are widths of the corresponding interchange ramps.

As shown in Figure 5.7, the shaded area roughly describes the right-of-way (ROW) area required to build the trumpet interchange. The following formula can be used to estimate the ROW area:

$$\begin{aligned} A_{IC_R} = & \left\{ \left[R_{EP1}^{\ 2} \tan\left(\frac{\pi - \theta_{EP}}{2}\right) \right] - \left[\frac{(\pi - \theta_{EP})}{2} \left(R_{EP1} - \left(\frac{d_{buf}}{2}\right) \right)^2 \right] \right\} \\ & + \left\{ \left[R_{EP2}^{\ 2} \tan\left(\frac{\theta_{EP}}{2}\right) \right] - \left[\frac{\theta_{EP}}{2} \left(R_{EP2} - \left(\frac{d_{buf}}{2}\right) \right)^2 \right] \right\} \\ & + \left\{ \left[\frac{1}{2} R_{EP4}^{\ 2} \tan(\theta_{EP4}) \right] - \left[\frac{\theta_{EP4}}{2} \left(R_{EP4} - \left(\frac{d_{buf}}{2}\right) \right)^2 \right] \right\} \\ & + \left\{ \begin{aligned} & \left[\frac{\theta_{EP3}}{2} \left(R_{EP3} + \left(\frac{d_{buf}}{2}\right) \right)^2 \right] \\ & - \left[\frac{l_{offset}}{2} \left(R_{EP3} \cos\left(\theta_{EP} - \frac{\pi}{2}\right) + l_{offset} \tan(\theta_{EP4}) \right) \right] \end{aligned} \right\} \end{aligned} \tag{5.40}$$

where: A_{IC_R} = Trumpet interchange right-of-way area (sq.ft) required

The first two terms of equation (5.40) represent the ROW areas required for constructing ramps 1 and 2 (two ramps placed in right and left sides of the new highway alignment, respectively), and the last two terms stand for those required for building ramps 3, 4, and 5; note that ramp numbers correspond to notations of their center points and arc distances. The estimated interchange ROW areas are then used to calculate the alignment ROW cost based on Jha's (2000) method, which is presented in equations (7.4) (see Section 7.1).

The earthwork cost of the trumpet interchange can be estimated with equation (7.6) (see Section 7.1), which is developed for that of the highway basic segments. The construction cost of a small bridge for grade separation in the trumpet interchange is estimated below.

Small Bridge Construction Cost for Grade Separation

A small bridge structure is used for grade separation where two highways cross each other. This structure can be used just for a grade separation purpose without any connection to the highways being crossed, or as a part of an interchange structure connecting the highways.

Normally for a small highway bridge, only 0 to 4 piers support its spans, and they are equally spaced. For instance, fewer than 3 piers may suffice for supporting a bridge over-passing a 2-lane highway. In addition, the pier heights may be considerably shorter than those of bridges crossing rivers. The pier heights of a small highway bridge follow the minimum vertical clearance recommended by AASHTO (2001).

Several cost models for (small and simple) highway-bridges are reviewed (rather than those for complex and huge bridges) in order to use them for estimating the grade separation cost of the trumpet interchanges. Note that among many types of highway bridges (such as stone and concrete girder bridges, reinforced concrete bridges, wooden bridges, metal truss bridges, stone and metal arch bridges, and suspension bridges), steel and concrete composite girder bridges have been most commonly employed as highway bridges (Kim, 2000).

Menn (1990) explored the cost of highway bridges using a sample of 19 highway bridges[12] built in Switzerland. He broke down the total bridge construction into three sub components: mobilization, structure, and accessories. According to the Menn's research, the mobilization is defined as the work required before construction can begin (e.g., cost for providing access to the construction site, preparation of site facilities, and procurement of equipment) and accessories include bearings, expansion joints, drainage system, guardrails, deck waterproofing system, and wearing surface. Figure 5.8 presents the contribution of those three components to the total construction cost of the 19 bridges by (Menn, 1990).

As shown in Figure 5.8, the bridge structure cost accounts for 78% of the total bridge cost, and costs of accessories and mobilization are 14% and 8% respectively, on average. Such result may be useful for approximately estimating total bridge construction cost if only the bridge structure cost is available; note that most bridge

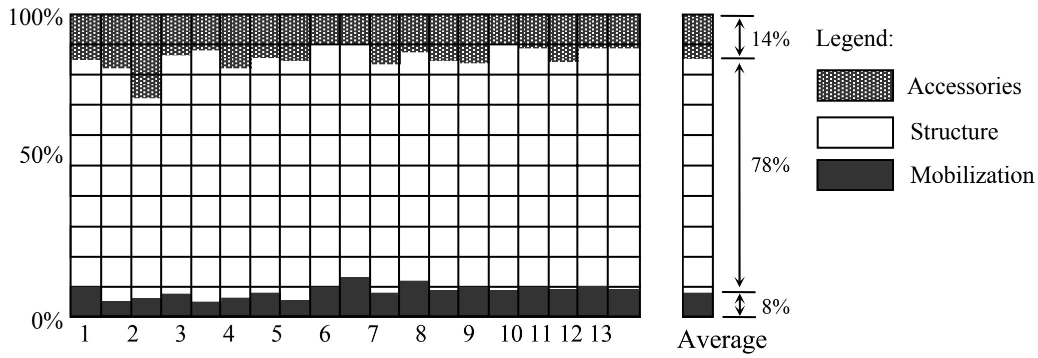
[12] The 19 samples used in Menn (1990) are steel-concrete composite bridges (mixed with concrete, reinforcing steel, and pre-stressing steel); among them, bridges 1 through 4 are elevated highways in urban areas, bridges 5 through 11 are viaducts in mountainous terrain, and bridges 12 through 19 cross valleys.

models are developed for estimating only the structure cost. Given the estimated bridge structure cost, the total bridge construction cost then can be expressed as:

$$C_{BR} = \frac{100}{78} \times C_{BS} \tag{5.41}$$

where: C_{BR} = Total bridge construction cost

C_{BS} = Bridge structure cost



Source: Menn (1990)

Figure 5.8 Costs of Mobilization, Structure, and Accessories as Percentages of Total Bridge Construction Cost

In most bridge models, normally many variables (such as number of spans, span lengths, volume and weight of concrete in slab) are used for estimating the bridge structure costs. However, a simple functional form of the model is preferable for the preliminary engineering cost estimation purpose (i.e., from the highway planning point of view). Here, we introduce two simple but useful models for estimating the bridge structure cost.

The first is O'Connor's (1971) theoretical model, which is also used in Kim's (2001) model, and the other is the model used by the Virginia Department of

Transportation (VDOT, 2003). O'Connor's model consists of two linear cost functions for superstructure and substructure of girder type bridges:

$$C_{BS} = \left[C_{BR_i^U} + C_{BR_i^L} \right] = \left[\left(\sigma_{1i} + \sigma_{2i} L_{spi} \right) + \left(\sigma_{3i} + \sigma_{4i} L_{spi} \right) \right] \tag{5.42}$$

where: C_{BS} = Bridge structure cost

C_{BR^U} = Bridge superstructure cost

C_{BR^L} = Bridge substructure cost

L_{sp} = Bridge span length

σ_1 and σ_2 are coefficients of bridge superstructure cost

σ_3 and σ_4 are coefficients of bridge substructure cost

In O'Connor's model, the bridge superstructure coefficients, σ_1 and σ_2 are differentiated by girder spacing, and those of substructure coefficients, σ_3 and σ_4 differ for pier heights. A detailed bridge cost formulation above and its implementation algorithm to the alignment optimization model may be found in Kim (2001).

Kyte et al. (2003) recently proposed a simple regression model for estimating preliminary engineering costs of highway bridges. This bridge model is originally designed for use it in Highway Construction Project Cost Estimation Tool (HCPCE) in VDOT. The functional form of this model is as follows:

$$C_{BS} = \left[91.3 l_{BGi} W_{Bi} + 68851.8 \right] \tag{5.43}$$

where: l_{BG} = Bridge length (ft)

W_B = Bridge width (ft)

In the model of Kyte et al. (2003), only two parameters representing length and width of a bridge are used. Thus, it is simple and sufficient for preliminary engineering purposes. This dissertation adopts that bridge model for estimating the grade separation cost of the trumpet interchange. Figure 5.9 presents the unit bridge cost ($/deck area) estimated with the model in 2004 US dollars.

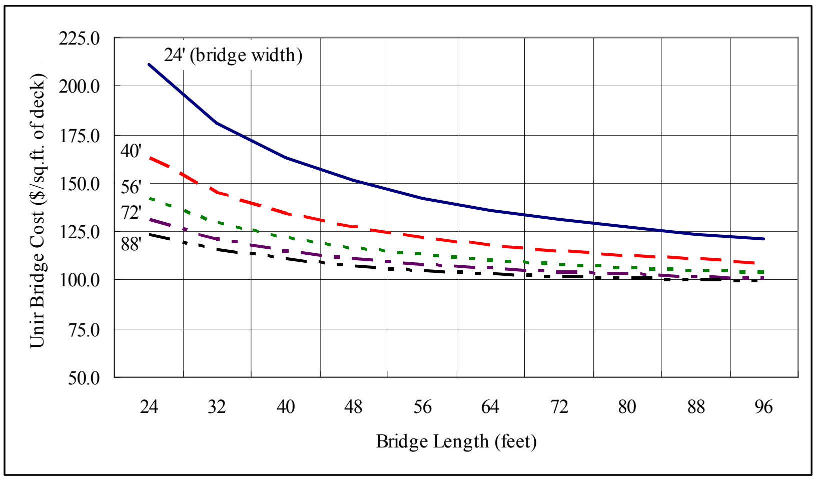


Figure 5.9 Unit Bridge Costs Estimated with Bridge Lengths and Widths

Bridge Length Estimation

In this section, we calculate the length of the small highway bridges in order to estimate the grade-separation cost of the small-scale interchanges. As shown in Figure 5.10, two types of grade separation structures might be considered as small bridges; a small bridge where a new alignment overpasses an existing road is shown in Figure 5.10 (a), and Figure 5.10 (b) presents a bridge on the existing road where the new highway is under-passed.

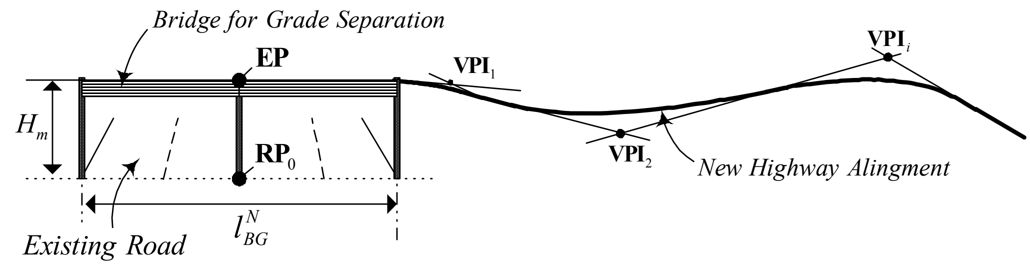


Figure 5.10 (a) New Highway Alignment Over-passing an Existing Road

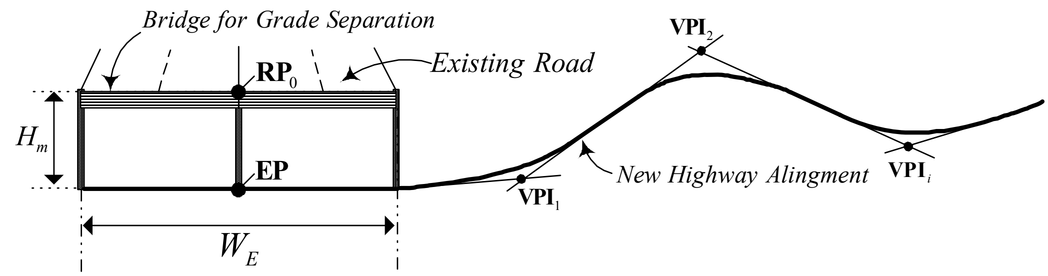


Figure 5.10 (b) New Highway Alignment Under-passing an Existing Road

In the figures, a minimum vertical clearance (H_m), two reference points (**EP** and **RP**$_0$), and several vertical points of intersections (*VPI*s) of the new alignment are used to draw its vertical profile. Lengths of bridges on a new alignment and on an existing road can be formulated as follows:

$$l_{BG}^{N} = l_{BG1}^{N} + l_{BG2}^{N} = \left[\frac{W_E + 2H_m / s_f}{\cos\left(\theta_{EP} - \pi / 2\right)} \right] + \left[W_N \tan\left(\theta_{EP} - \frac{\pi}{2} \right) \right] \tag{5.44a}$$

$$l_{BG}^{E} = l_{BG1}^{E} + l_{BG2}^{E} = \left[\frac{W_N + 2H_m / s_c}{\cos\left(\theta_{EP} - \pi / 2\right)} \right] + \left[W_E \tan\left(\theta_{EP} - \frac{\pi}{2} \right) \right] \tag{5.44b}$$

where: l_{BG}^{N} = Length (ft) of highw-bridge on a new alignment; refer to Figures 5.10 (a) and 5.12 (a)

l_{BG}^{E} = Length (ft) of highw-bridge on an existing road; refer to Figures 5.11 (b) and 5.12 (b)

W_E = Width (ft) of the existing road

W_N = Width (ft) of the new alignment

s_f = Fill slope required for overpass-bridge construction;

s_c = Cut slope required for underpass-bridge construction;

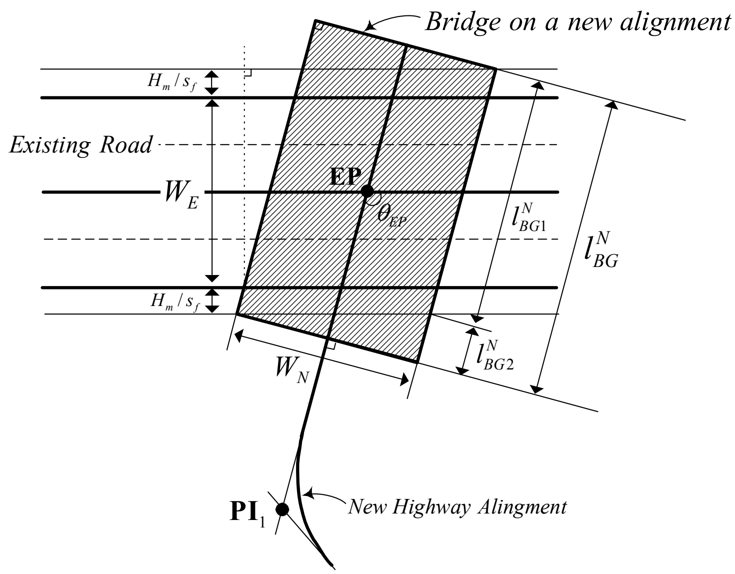


Figure 5.11 (a) Highway Bridge on a New Alignment

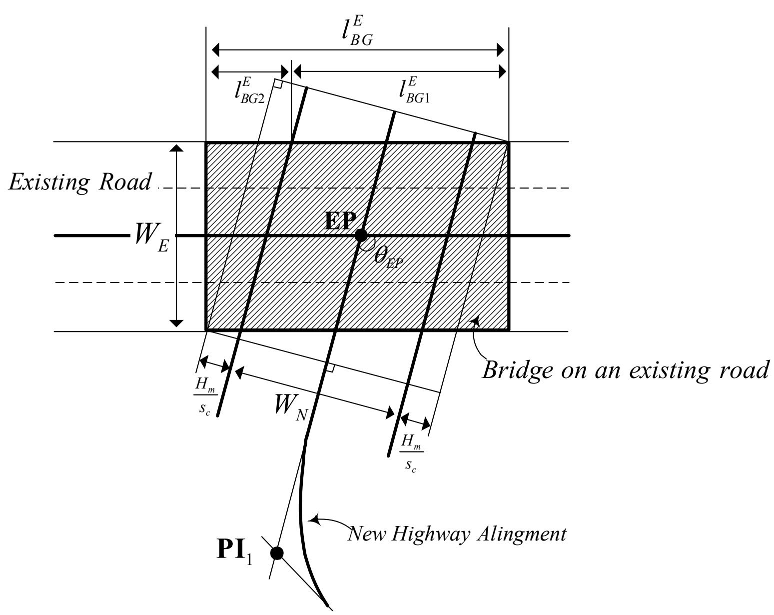


Figure 5.11 (b) Highway Bridge on an Existing Road

As shown in Figure 5.11, the bridge length varies depending on various design inputs associated with the existing and new highways and the intersection angle (denoted as θ_{EP}). The design variables are obtained from the model inputs and the highway crossing angle, θ_{EP} can be computed with equation (5.16). To better understand the computation of the length of highway bridges in equation (5.44), readers may refer to the cross-sectional views shown in Figure 5.12.

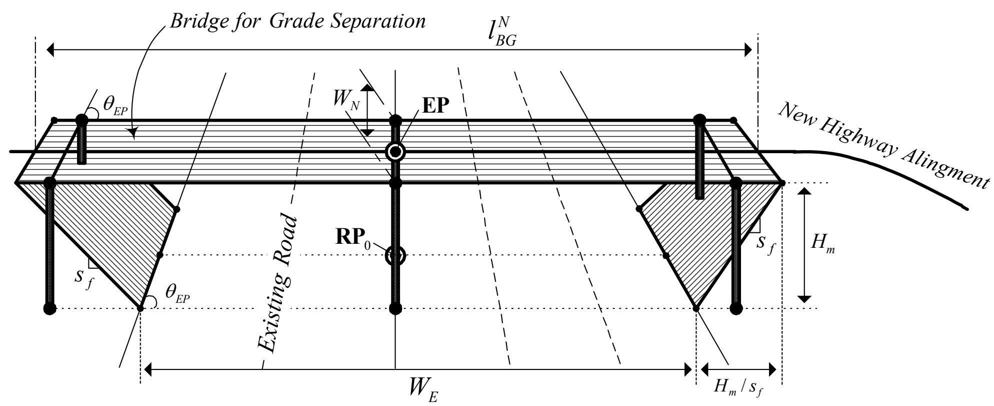


Figure 5.12 (a) Cross Sectional View of Highway Bridge on a New Alignment

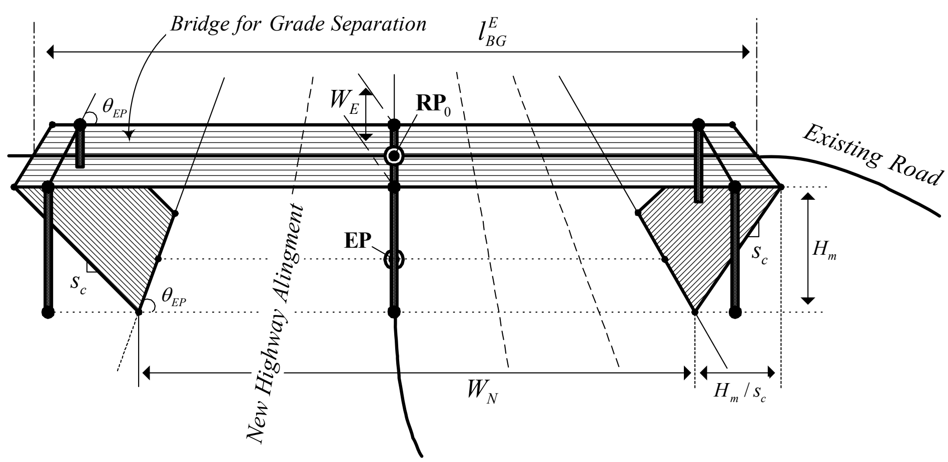
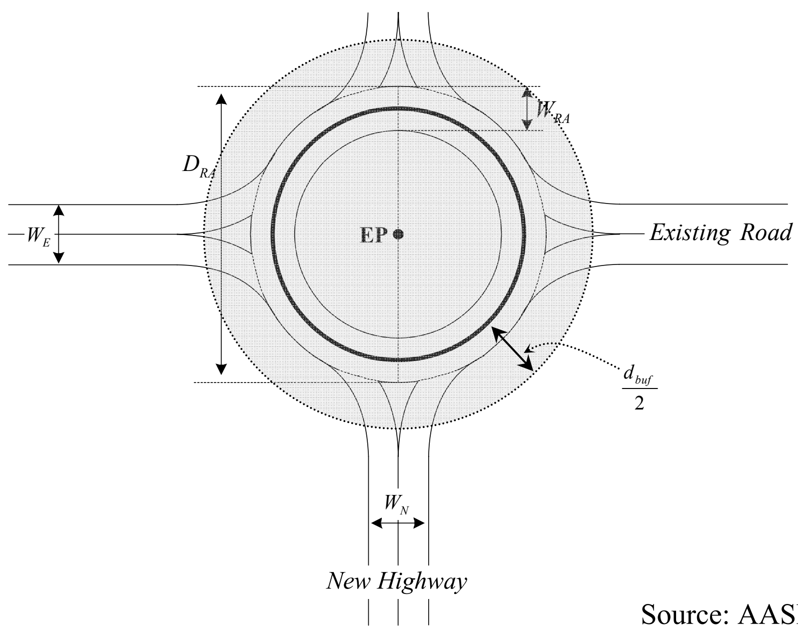


Figure 5.12 (b) Cross Sectional View of a Highway Bridge on an Existing Road

5.2.2.3 Roundabouts

Fewer roundabouts are considered in highway engineering compared to many other highway cross-structures, such as the typical intersections and interchanges described earlier. However, many modern roundabouts have been constructed recently in the United States as well as in many other countries (AASHTO, 2001). Figure 5.13 shows a typical geometric configuration of a modern roundabout considered here.



Source: AASHTO 2001

Figure 5.13 Geometric Configuration of a Typical Modern Roundabout

A mathematical formulation for estimating construction cost of the roundabout is relatively simple compared to the other structures described earlier. Given the inscribed circle diameter and circulatory roadway width of the roundabout (denoted as D_{RA} and W_{RA} respectively shown in Figure 5.13) and location of the highway endpoint (**EP**), the roundabout pavement-cost function can be described as follows:

$$C_{RA_P} = R_{RA} \times K_p = \left[\frac{1}{2} \times \left(D_{RA} - W_{RA}\right)\right] \times K_p \tag{5.45}$$

where: C_{RA_P} = Roundabout pavement cost

R_{RA} = Roundabout radius; R_{RA}=$1/2\left(D_{RA} - W_{RA}\right)$

The value of the inscribed circle diameter (D_{RA}) used in equation (5.45) can be found in AASHTO (2001); "Modern roundabouts range in size from mini-roundabouts with inscribed circle diameters as small as 15m (50ft), to compact roundabouts with inscribed circle diameters between 30 and 35m (98 to 115ft), to large roundabouts, often with multilane circulating roadways and more than four entries up to 150m (492ft) in diameter." (AASHTO, 2001)

For estimating the roundabout ROW cost, the roundabout boundary should be specified; it can be defined with the circle diameters (D_{RA}) and the location of the highway endpoint (**EP**). The shaded in Figure 5.13 approximately describes the area required for constructing the roundabout, which can be estimated as:

$$A_{RA_R} = \pi \times \left(R_{RA} + \frac{d_{buf}}{2}\right)^2 = \pi \times \left(\frac{D_{RA} - W_{RA} + d_{buf}}{2}\right)^2 \tag{5.46}$$

where: A_{IC_R} = Right-of-way area (sq.ft) required for roundabout construction

d_{buf} = Buffer width (ft) required for roundabout construction; $d_{buf} \geq W_{RA}$

As in the other structures, the roundabout ROW cost can also be estimated with equations (7.4). Note that equations (5.23) and (5.24) can also be used for estimating the roundabout earthwork cost, given the ground elevations of the roundabout area.

Chapter 6: Alignment Optimization for a Simple Highway Network

6.1 Introduction

The new highway addition to an existing road network is normally intended to improve service times for the network users (drivers). Such a supply action basically provides additional capacity of the network for dealing with traffic. The main objective in the proposed optimization model, as stated in Chapter 1, is to find the alignment of a new highway that best improves the traffic performance of the system, while considering various construction costs and constraints associated with the project. To accomplish this objective, this dissertation jointly considers (in the optimization process) user cost savings from the highway addition as well as its construction costs.

A bi-level programming structure is proposed to solve the complex problem. The alignment optimization and traffic assignment problems are formulated as the upper-level and lower-level problems of the bi-level structure, respectively. The basic model structure of the bi-level programming problem and optimization procedure are proposed in Section 6.2. Inputs required for the traffic assignment problem are presented in Section 6.3. Representation of a given highway network and input O/D trip matrix are also discussed in that section.

6.2 Basic Model Structure

In order to solve the complex optimization problem, we start with a generalized formulation. As discussed earlier, we formulate the problem as a bi-level programming problem. The upper-level problem is the highway alignment optimization (HAO) problem, and the lower-level one is the deterministic (and static) traffic assignment problem. In the proposed model, the equilibrium traffic flows found from the assignment process will be used for estimating the user cost components of the upper-level objective function during the optimization process.

Three types of decision variables are used in the model framework: (i) point of intersections (**PI's**) of the candidate alignment, (ii) its endpoints (**EPs**), and (iii) traffic flows (**x**) operating on the network. Cost functions comprised in the upper-level objective function are sensitive to these variables, and they are formulated as functions of **PI's** and **EPs** indirectly.

The bi-level programming problem described below shows the basic model structure of the proposed network problem. Readers may also refer to Figure 6.1 (the optimization procedure) to help understand the concept of the model structure.

HAO Problem ***(Upper-Level Problem)*****; see Section 6.2.1**

$$\text{Minimize } Z_{UL} = C_{User} + C_{T\_Agency} + C_{Environ} \tag{6.1}$$

$$\text{subject to: } \begin{cases} \text{1. Design constraitnts for horizontal and vertical alignments} \\ \text{2. Environmental and geographical constraints} \end{cases}$$

where: C_{T_Agency} = Total agency cost associated with the new highway construction

C_{User} = Total network user cost from the new highway addition

$C_{Environ}$ = Total environmental cost (=$\sum C^{i}_{Environ}$);

Note that C_{User} is function of traffic flow (x), and x is implicitly defined by the following lower-level problem:

Traffic Assignment Problem ***(Lower-Level Problem)*****; see Section 6.2.2**

$$\text{Minimize } Z_{LL} = \begin{cases} \sum_a \int_0^{x_a} t_a(x_a)\, dx_a & \text{for User Optimum} \quad (6.2a) \\ \sum_a x_a t_a(x_a) & \text{for System Optimum} \quad (6.2b) \end{cases}$$

subject to:

$$\sum_k f_k^{rs} = q_{rs} \qquad \forall r,\ s \tag{6.3}$$

$$f_k^{rs} \geq 0 \qquad \forall k,\ r,\ s \tag{6.4}$$

$$x_a = \sum_r \sum_s \sum_k f_k^{rs} \delta_{a,k}^{rs} \qquad \forall a \in \boldsymbol{A} \tag{6.5}$$

where $\boldsymbol{A}$ = Arc (index) set of a given highway network; $a \in \boldsymbol{A}$

x_a = Flow on arc a; $\mathbf{x} = (\ldots,\ x_a,\ \ldots)$

t_a = Travel time on arc a; $\mathbf{t} = (\ldots,\ t_a,\ \ldots)$

f_k^{rs} = Flow on path k connecting O/D pair r - s; $\mathbf{f^{rs}} = (\ldots,\ f_k^{rs},\ \ldots)$

q_{rs} = Trip rate between origin r and destination s; $r \in \boldsymbol{R}$, $s \subseteq \boldsymbol{S}$

$\boldsymbol{N}$ = Node (index) set of a given highway network; $\boldsymbol{R} \subseteq \boldsymbol{N}, \boldsymbol{S} \subseteq \boldsymbol{N}$

$\delta_{a,k}^{rs}$ = Indicator variable;

$$\delta_{a,k}^{rs} = \begin{cases} 1 & \text{if arc } a \text{ is on path } k \text{ between O/D pair } r\text{ - }s \\ 0 & \text{otherwise} \end{cases}$$

$\boldsymbol{R}$ and $\boldsymbol{S}$ = Sets of origin and destination nodes, respectively

6.2.1 HAO Problem (Upper-Level Problem)

The formulation of the HAO problem includes an objective function and constraints associated with highway construction. We define the objective function (Z_{UL}) is sum of (i) the total user cost (C_{User}) from the new highway addition to the existing network, (ii) the total agency cost (C_{T_Agency}), and total environmental cost ($C_{Environ}$) accompanied by construction of the highway alignment. As shown in equation (6.1), the problem is formulated to minimize Z_{UL} of the candidate alignment, while satisfying the alignment-sensitive constraints.

Note that to ensure compliance with the specified constraints, penalty costs (C_P) may be added in the objective function. Mathematical formulation of a penalty function (called the soft penalty) is described in Chapter 4. Coupled with the penalty function, the equation (6.1) becomes:

$$\text{Minimize } Z_{UL} = C_{User} + C_{T\_Agency} + C_{Environ} + \sum C_P \tag{6.6}$$

where: $\sum C_P$ = Total penalty associated with the model constraints

Various cost components included in C_{User} and C_{T_Agency} are precisely formulated in Chapter 7 separately.

Constraints of the HAO Problem

Two types of model constraints are considered in the HAO problem. These are (i) design constraints and (ii) environmental and geographical constraints. The design constraints are used to insure that the candidate alignments satisfy AASHTO's (2001) design standards. The environmental and geographical constraints are used to represent subjective road-project issues (e.g., untouchable and preferred areas for right-of-way boundary of the new highway) that should be satisfied for all highway alternatives. These constraints are very sensitive to the topography of the project area, preferences of highway planners and designers, opinions from public hearings. Thus, the proposed optimization model is designed so that such subjective constraints are provided by the model users (i.e., user specifiable), while the design constraints are governed by AASHTO standards.

Table 6.1 Constraints for Highway Alignment Optimization Problem

<table>
<tr><th colspan="2">Category</th><th>Type of Constraints</th></tr>
<tr><td rowspan="2">Design Constraints</td><td>Horizontal Alignment</td><td>1. Minimum horizontal curvature constraint
2. Minimum superelevation runoff length
(only if transition curves are used)</td></tr>
<tr><td>Vertical Alignment</td><td>3. Minimum vertical curvature constraint
4. Maximum gradient constraint
5. Minimum vertical clearance for fixed points</td></tr>
<tr><td colspan="2">Environmental and Geographic Constraints</td><td>6. Environmentally sensitive areas
7. Areas outside interest</td></tr>
</table>

Note: constraints 1 to 3 are controlled by P&R presented in Chapter 4, and constraints 4 to 7 are controlled by FG methods in Chapter 3.

Table 6.1 summarizes the alignment constraints considered in the upper-level (the HAO) problem. The minimum horizontal curvature constraint, which is the first constraint in the table, depends on design speed, supperelevation, and coefficient of side friction of the new highway (see AASHTO, 2001). The second one, the minimum superelevation runoff length depends on superelevation, maximum relative gradient (in percent), number of lanes rotated, and width of the road (see Section 5.2 and AASHTO, 2001).

The third constraint, the minimum vertical curvature constraint, restricts the length of crest vertical curves to be met with the vertical sight distance as well as guarantees headlight distance and motorist comfort on sag vertical curves. The equations used for computing this constraint can be found in Jong (1998) or AASHTO (2001). Note that in the model, the solution alignments that violate any of these three constraints, which are associated with either horizontal curved sections or vertical curved sections of the alignments, are processed with the P&R method discussed in Chapter 4.

The fourth constraint, the maximum gradient constraint, normally depends on the nature and importance of the new highway, design speed, and topography of the study area (AASHTO, 2001), and it is specified by the model users.

The fifth constraint, minimum vertical clearance is used for restricting road elevation where the new highway intersects existing roads or rivers. For grade separation with an existing road, the minimum required elevation difference between the new and existing roads may be found in AASHTO (2001); however, that required for a bridge crossing a river may vary depending on its water level information (e.g., 100 year floodplains). Note that the vertical feasible gate (VFG) approach, presented in Section 3.3, is designed to represent these gradient and fixed-point constraints in the optimization process.

The environmental and geographical constraints (sixth and seventh in Table 6.1) are too complex to formulate with any single mathematical form since they vary subjectively for various user specifications. When considering roadway construction in a given project area, various geographically sensitive regions (such as historic sites, flood plains, and public facilities) may exist. These control areas should be avoided by the new alignment and to the extent possible, its impact to these regions should be minimized. A way of representing such complex constraints in a machine readable format is presented in Section 3.2 by developing horizontal feasible gate (HFG) approach.

Additionally, the candidate alignments generated from the model also definitely satisfy (8) the alignment boundary conditions, (9) the alignment necessary conditions, (10) the continuity condition, and (11) continuously differentiable condition besides the seven distinct constraints above; please refer to Jong (1998) for detailed discussion of these additional conditions.

6.2.2 Traffic Assignment Problem (Lower-Level Problem)

It is assumed that the network drivers adjust their travel paths with response to various network configurations due to the addition of different candidate alignments. The traffic assignment problem, which is considered as the lower-level problem in the model, is designed to represent such a traffic effect in the evaluation process. Equilibrium traffic flows for the changing highway network are estimated from the traffic assignment process in the model, and they are ultimately used for computing costs associated with user travels in the upper-level problem.

User and System Optimal Traffic Assignment Problems

Typically, two assignment principles are used in the assignment problem for representing interaction between supply (network design) and demand (network users) actions. These are *user optimal* (UO) and *system optimal* (SO) principles:

User Optimal (UO):	O/D flows are assigned to possible paths in the network with minimum travel time.
System Optimal (SO):	O/D flows are assigned such that total travel time in the network is minimized.

The static UO assignment problem is to find the arc flows, **x,** that satisfy the user equilibrium criterion when all the origin-destination entries, $q_{rs}\ \forall r,s\ (r \neq s)$ have been appropriately assigned (Sheffi, 1984). This equilibrium arc-flow pattern can be obtained by solving equation (6.2a), originally proposed by Beckmann et al. (1956), subject to three types of constraints: (i) flow conservation constraints (equation (6.3)), (ii) non-negativity constraints (equation (6.4)), and (iii) incident relationships between arc and path flows (equation (6.5)). Note that the UO objective function (equation (6.2a)) is strictly convex everywhere (in **x)** for the static traffic assignment problem (Sheffi, 1984); thus, it has a unique solution.

The SO assignment problem is to find the arc flows, **x** that minimize total travel time of the network subject to the same constraints as in the UO assignment problem. The SO objective function (equation (6.2b)) is also strictly convex in **x** for the same criteria as in the UO problem and may be rewritten as the following formulation (Sheffi, 1984 and Thomas, 1991):

$$\text{Min } Z(\mathbf{x}) = \sum_a \int_0^{x_a} \left(\frac{d\left(x_a t_a(x_a)\right)}{dx_a} \right) dx_a \tag{6.7}$$

The solution from the SO assignment problem with equation (6.7) indicates that all used paths have equal and minimum marginal-travel times between any O/D pair, while that from the UO problem indicates that all used paths have equal and minimum travel time between any O/D pair (Thomas, 1991). Note that a decision on which assignment principle (between UO and SO) is used is user-specifiable in the proposed optimization model.

A static (and deterministic) traffic assignment method is adopted in this dissertation since it is commonly used in many planning applications; the assignment model, although it has a limitation in capturing traffic phenomena such as the propagation of shockwaves and queue spillovers, is widely used by agencies for planning applications from infrastructure structure improvement to traffic maintenance and congestion management.

Convex Combinations Method for Static Traffic Assignment Problems

The convex combinations method, originally developed by Frank and Wolfe in 1956, has been widely used for solving quadratic programming problems with linear constraints. The method is also known as the Frank-Wolfe algorithm and is very useful for general application of the static traffic assignment problems; both the UO and SO traffic assignment principles are applicable to the well known iterative algorithm. Starting with a feasible solution (a set of arc flows $\{x_a^0\}$), the Frank-Wolfe algorithm will converge after a finite number of iterations. The following steps specify the basic Frank-Wolfe algorithm (Sheffi, 1984; Thomas, 1991):

STEP 1: Initialization

(a) Set iteration counter $n = 0$.

(b) Set $t_a = t_a(0)$, $\forall a \in \boldsymbol{A}$; $t_a(0)$ is usually the free-flow travel time on arc a.

(c) Assuming travel times $\{t_a = t_a(0)\}$, perform all-or-nothing assignment of the O/D matrix. Let y_a^0 be the flow assigned to arc a.

(d) Set $x_a^0 = y_a^0$.

STEP 2: Update arc travel time

(a) Set $n = n + 1$.

(b) Set $t_a^n = t_a^{n-1}\left(x_a^{n-1}\right)$, $\forall a \in \boldsymbol{A}$

STEP 3: Determination of the auxiliary flows $\{y_a^n\}$

Perform all-or-nothing assignment of the O/D matrix on the basis of travel times $\{t_a^n\}$ in order to obtain $\{y_a^n\}$ which is a set of auxiliary link flows required for finding the decent search direction.

STEP 4: Adjustment of the assigned arc flows (line search)

Set $x_a^n = x_a^{n-1} + \lambda^n\left(y_a^n - x_a^{n-1}\right) \quad \forall a \in \boldsymbol{A}$

where λ^n is chosen so as to minimize:

$$Z_{LL}^n = Z_{LL}\left(\lambda^n\right) = \begin{cases} \sum_a \int_0^{x_a^n} t_a(x_a)\, dx_a, & \text{if UO traffic assignment} \\ \sum_a \int_0^{x_a^n} \left(\dfrac{d\left(x_a t_a(x_a)\right)}{dx_a}\right) dx_a, & \text{if SO traffic assignment} \end{cases}$$

$$, \text{for } 0 \le y_a^n \le 1$$

STEP 5: Convergence test

If a convergence criterion is satisfied, stop (the current solution, $\{x_a^{n+1}\}$ is the set of UO or SO arc flows); otherwise go to step 2.

The optimal move-size (λ^n) at every iteration in the Frank-Wolfe algorithm may be found with either *Bisection* or *Golden Section* method in general; we employ the *Golden Section* method in the static traffic assignment program. Detailed illustrations of those line search methods are provided in many textbooks on numerical optimization.

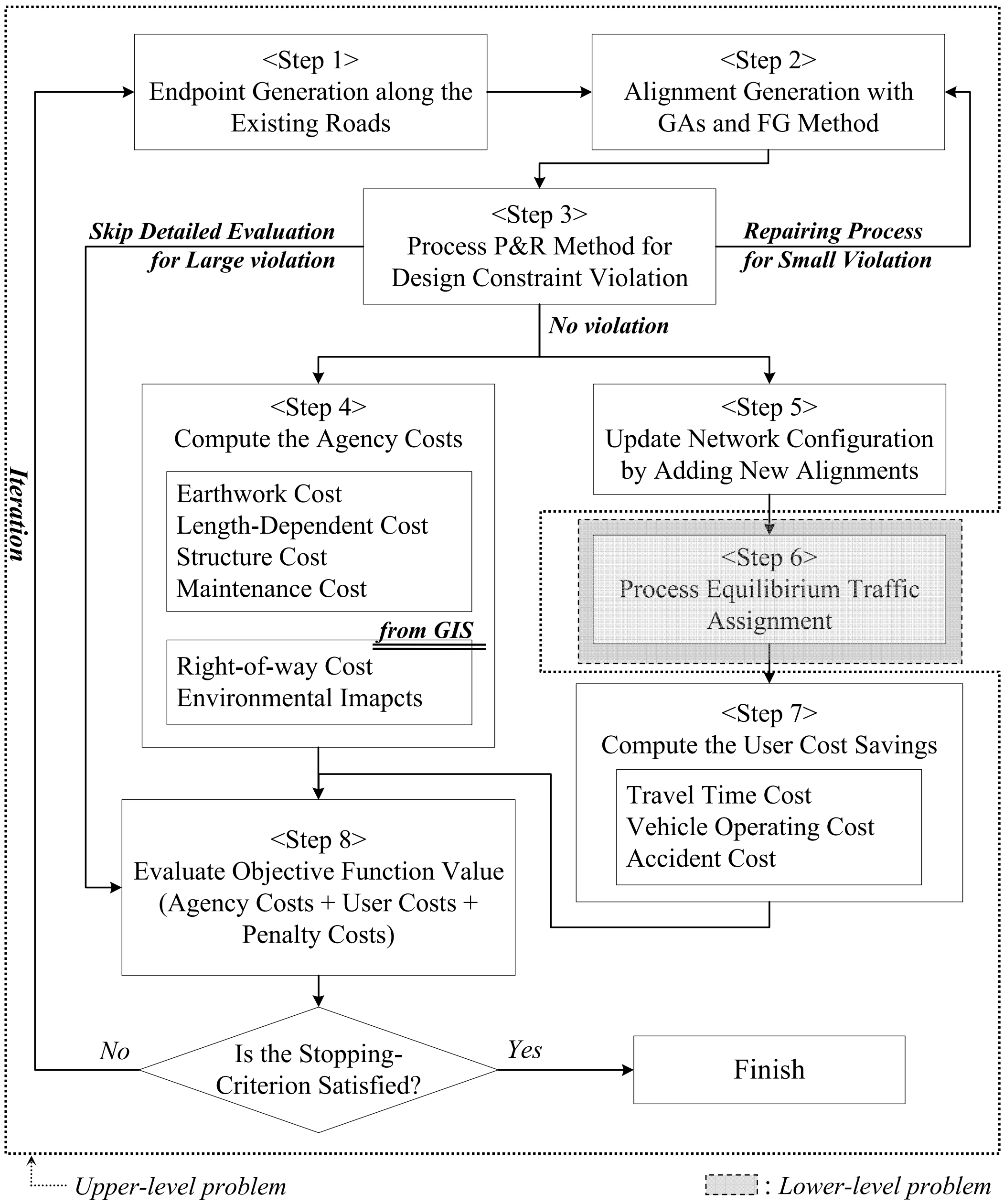


Figure 6.1 Optimization Procedure of the Proposed Network Model

6.2.3 Optimization Procedure

The optimization procedure of the proposed model consists of eight steps, as shown in Figure 6.1. These are (1) *endpoint generation process* (see section 5.1), (2) *alignment generation process* (see sections 5.2 and 3.2), (3) *prescreening & repairing (P&R) process* (see section 4.2), (4) *agency cost evaluation* (see section 7.1), (5) *network-configuration update* (refer to section 6.3), (6) *equilibrium traffic assignment process* (see section 6.2.2), (7) *user cost evaluation* (see section 7.2), and finally (8) *total cost evaluation process* (refer to section 6.2.1).

In step 1, *the Endpoint Determination Procedure (5.1)* is used to generate the endpoints of a new highway. The highway endpoints are determined on the specified road segments or on a set of pre-defined candidate points. In step 2, the horizontal and vertical alignments of the new highway are simultaneously produced with a set of PI's generated from the genetic operators (adopted from Jong and Schonfeld, 2003) and FG approach. The horizontal alignment is generated through the *Horizontal Alignment Generation Procedure (5.2)* with the set of PI's, and the vertical alignment is also created (jointly) with the PI's, while their elevations (i.e., Z values of PI's) are obtained from the *Road Elevation Determination Procedure (3.2)* presented in Section 3.3. After the road generation procedures are completed, the alignments are subjected to the P&R process in step 3 to determine whether they are sent to the repairing process, to prescreening process, or to the normal evaluation procedure. Note that if there is sufficient room for repairing design-constraint violations in the alignments, their PI's are repetitively manipulated through the repairing process until they are completely fixed and then they are returned to step 3; however, if there is a large design violation in the alignments, a penalty is assigned to their objective function value and the detailed evaluation procedures are skipped. If they have no design violations, the alignments go to the next step.

In step 4, alignment agency costs (such as earthwork, right-of-way, length-dependent, structure costs) are computed (see Section 7.1). Here, a GIS module is used for computing the right-of-way cost and the alignment's environmental impacts to the study area. In step 5, the configuration of the given road network is updated with the addition of various generated highway alignments; for instance, road length (of the new alignment as well as of existing roads) and the incidence matrix of the given road network, which are used for inputs of the traffic assignment process, are updated. Following the step 5, the traffic assignment module is processed for estimating the equilibrium traffic flows of the network (in step 6). In step 7, three types of user cost components (travel time cost, vehicle operating cost, and accident cost) are computed based on the outputs from the assignment process; the user cost savings (user cost difference between before and after the new alignment addition) are also computed in step 7. Finally, in step 8 the model evaluates the objective function value (i.e., sum of the all cost components including user, agency, and penalty costs) of the candidate alignment with the genetic algorithms (GAs).

6.3 Inputs Required for Traffic Assignment

Large amounts of input data are required to perform a traffic assignment analysis for a highway network. These include typically (i) the physical layout of the highway network (e.g., highway type, length, capacity, and free-flow speed and location of highway junction points), (ii) trip rates between origin and destination nodes (i.e., O/D trip matrix) of the network, and (iii) travel time functions (known as link performance or volume-delay functions) for estimating travel time of the network travelers. This section describes those input requirements.

6.3.1 Representation of Highway Network and O/D Trip Matrix

In the traffic assignment problem, a real highway network can be represented as a directed graph consisting of a finite set of nodes (or points or vertices) and pairs of which joined by one or more arcs (or links) in the network. Figure 6.2 shows representation of a typical highway network used for the traffic assignment process. In the figure, numbers marked with an italic font stand for a set of arcs, and those with boldface depict a set of nodes.

Normally, two subsets of nodes are used for the network representation. The first one is a set of origin and destination points (known as centroids) at which all trips are assumed to start and finish. The other one is a set of junction nodes representing points at which highways intersect each other (e.g., interchanges and intersections) or points at which physical nature of a highway is remarkably changed (e.g., highway-capacity-increase points due to increase in number of lanes). Dummy nodes may also be used to more realistically represent a highway junction node by breaking up it into several dummy points; typically more dummy nodes are used for more microscopic network representation.

Arcs are basically one-way sections of highways, and are typically identified by their start and end points (called initial nodes and final nodes, respectively). Two types of arcs are normally used for representing the highway network: (i) centroid connectors and (ii) highway arcs. Centroid connectors are not actual roads but conceptual representations of arcs that connect the centroids (trip origin and destination points) and the highway junction nods. Highway arcs connect the highway junction nodes, and can be classified into several sets of categories based on their speeds and capacities and access control types designed. In our traffic assignment problem, four types of highway arcs are used with different levels of the design characteristics; they are freeways, expressways, arterials, and collectors (refer to Table 6.2). Additionally, dummy arcs may also be used for representing the highway network by connecting dummy nodes at both of their ends (readers may refer to Figure 6.4). More detailed discussion of the node and arc (link) representation for traffic assignment problems may be found in many related studies such as Sheffi (1984) and Thomas (1991).

Let us now suppose that a new highway alignment is generated from the *highway alignment generation procedure (5.2)*, connecting in the middle of existing highways *19-20* and *21-22* in the highway network shown in Figure 6.2. Then, the network might be modified as the one presented in Figure 6.3. Note that the new constructed alignment has its realistic horizontal and vertical profiles with various design specifications although it is just represented with two straight lines (arcs) in the figure. As shown in Figure 6.3 (compared to the network in Figure 6.2), the numbers of highway arcs and junction-nodes are increased due to the highway addition process, and their (ID) numbers and properties (e.g., lengths of highway arcs *19, 20, 21, 22, 23, 24, 25, 26, 27, and 28* and locations of highway nodes **507** and **508**) are also newly updated from that process.

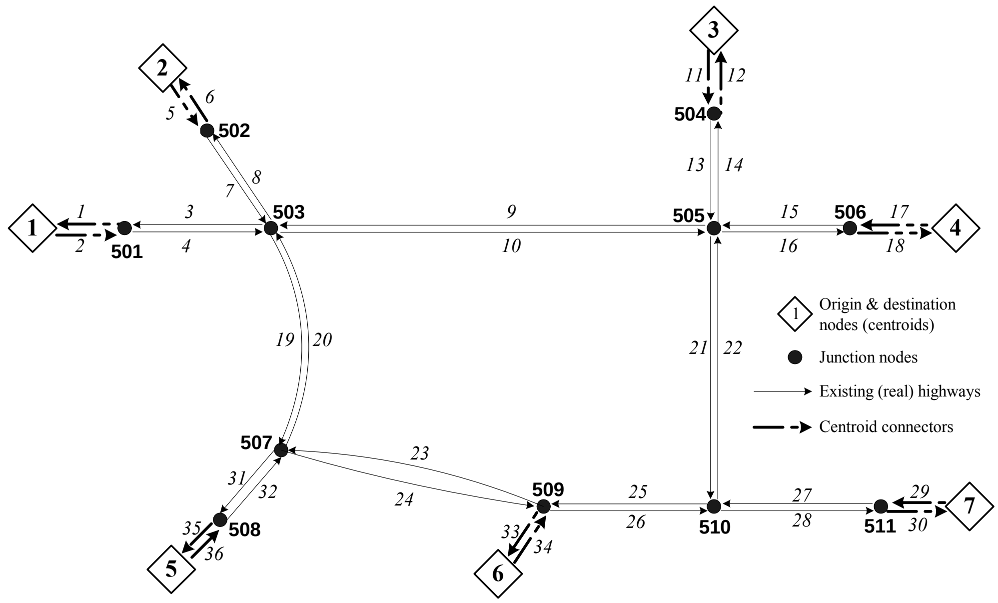


Figure 6.2 An Example Road Network Before a New Highway Addition

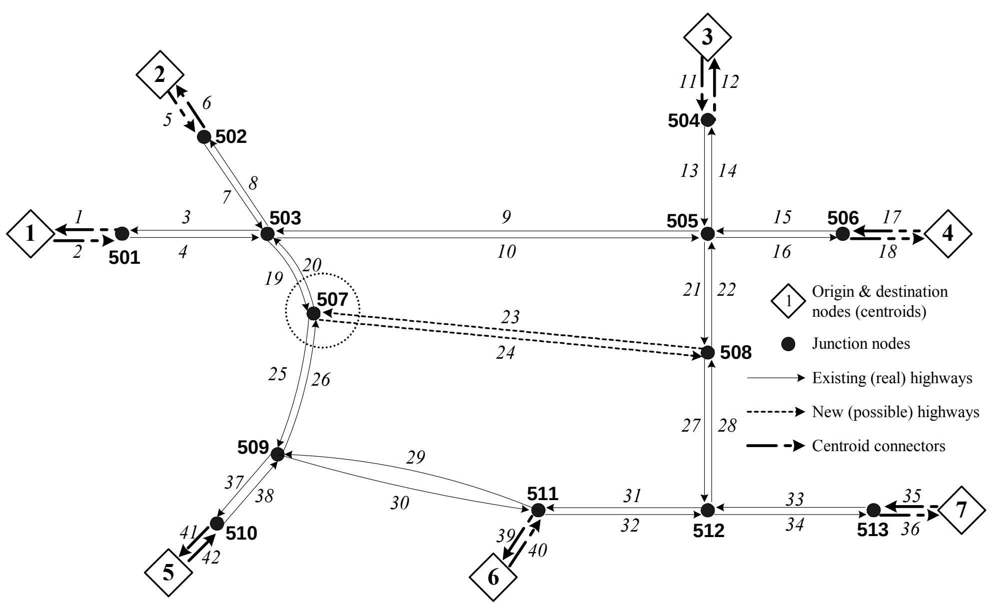


Figure 6.3 An Example Road Network After a New Highway Addition

Table 6.2 presents an example input layout of the arcs and arc properties (e.g., road type, length, capacity, speed, exit type, etc.) used in the traffic assignment process for the network updated from the highway addition. In the table, shaded rows indicate (i) the arcs of the new alignment added to the existing network and (ii) the arcs whose properties are updated from the highway addition. Note that they are iteratively updated whenever the new alignments are generated during the optimization process.

Table 6.2 Example Input Layout of a Highway Network for the Assignment Process

Arc number	Initial node	Final node	Road type	Length (feet)	Number of lanes	Capacity (vphpl)	Free-flow speed (mph)	Exit type	Cycle length (sec)	Effective Green (sec)
1	501	1	Centroid	0	2	99,999	65	0	-	-
2	1	501	Centroid	0	2	99,999	65	0	-	-
3	503	501	Freeway	13,411	2	2,200	65	0	-	-
4	501	503	Freeway	13,464	2	2,200	65	0	-	-
5	2	502	Centroid	0	2	99,999	50	0	-	-
6	502	2	Centroid	0	2	99,999	50	0	-	-
7	502	503	Arterial	16,632	2	1,800	50	0	-	-
8	503	502	Arterial	16,648	2	1,800	50	0	-	-
⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮
19	503	507	Arterial	8,131	2	1,800	45	0	-	-
20	507	503	Arterial	8,158	2	1,800	45	0	-	-
21	505	508	Freeway	11,616	2	2,200	65	0	-	-
22	508	505	Freeway	11,616	2	2,200	65	0	-	-
23	508	507	Arterial	18,110	2	1,800	65	0	-	-
24	507	508	Arterial	18,110	2	1,800	65	0	-	-
25	507	509	Arterial	11,405	2	1,800	45	0	-	-
26	509	507	Arterial	11,405	2	1,800	45	0	-	-
27	508	512	Freeway	9,821	2	2,200	45	0	-	-
28	512	508	Freeway	9,800	2	2,200	45	0	-	-
⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮	⋮
37	509	510	Arterial	7,530	2	1,800	45	0	-	-
38	510	509	Arterial	7,530	2	1,800	45	0	-	-
39	511	6	Centroid	0	2	99,999	50	0	-	-
40	6	511	Centroid	0	2	99,999	50	0	-	-
41	510	5	Centroid	0	2	99,999	45	0	-	-
42	5	510	Centroid	0	2	99,999	45	0	-	-

Exit type: 0 = grade-separated interchange, 1 = signalized intersection
2 = stop-controlled intersection, 3 = roundabout

Exit type in Table 6.2 is used for indicating the type of access-control designed at the final node of each highway arc (e.g., grade separated interchange and signalized intersection). For instance, if a highway arc is approaching a signalized intersection, additional information required for reflecting the intersection effect (such as cycle length and effective green time for traffic using that arc) would be needed for calculating the arc capacity. Note that in the proposed assignment problem, interchange junction-effects are assumed to be negligible; thus, an interchange can be represented with a single junction node (e.g., node **507** in Figure 6.3) while the at-grade intersection may be represented with several dummy nodes and arcs as shown in Figure 6.4.

Figure 6.4 shows a microscopic representation of junction node **507**, which is one of the alignment endpoints of the new highway added to the network. Such a detailed sub-network may be used in the network representation if a junction node is considered as an intersection node or if traffic delay at that node is particularly important in the traffic assignment process. Intersection volume-delay functions may be employed for estimating travel time of travelers on the dummy arcs while travel time functions (e.g., BPR functions) are used for travel time on other regular highway arcs.

Besides the network information stated above, specification of origin and destination (O/D) trip matrix is also required to perform the traffic assignment process. An example layout of input O/D trip matrix used in the assignment process for the example network shown in Figure 6.3 is illustrated in Table 6.3. As shown in the table, basically trip rates of origin-destination pairs and their peak hour durations are required to construct the input O/D matrix.

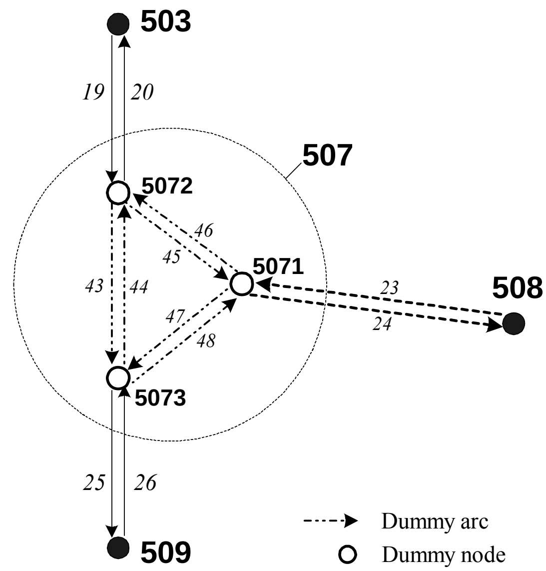


Figure 6.4 Microscopic Representation of a Highway Node

Table 6.3 Example Input Layout of O/D Matrix

Origin node	Destinatioin node	Average traffic volume (veh/hr)	Peak hour factor		Peak duration (hr)	
			AM peak	PM peak	AM peak	PM peak
1	***2***	85	0.9	0.8	3	3
1	***3***	70	1.1	1		
1	***4***	600	1.3	0.7		
1	***5***	200	1.3	0.7		
1	***6***	300	1.4	0.6		
1	***7***	1000	1.3	0.5		
.	.	.	.	.		
.	.	.	.	.		
.	.	.	.	.		
4	***1***	600	0.6	1.4		
4	***2***	400	0.7	1.3		
4	***3***	350	0.8	0.8		
4	***5***	80	1.2	0.7		
4	***6***	600	1.4	0.7		
4	***7***	600	1.3	0.8		
.	.	.	.	.		
.	.	.	.	.		
.	.	.	.	.		
7	***1***	1000	0.5	1.3		
7	***2***	400	0.6	1.4		
7	***3***	300	0.9	0.9		
7	***4***	800	0.7	1.3		
7	***5***	200	1	1		
7	***6***	200	1	1		

6.3.2 Travel Time Functions

Many travel-time functions (which are also known as link-performance functions or volume-delay models) have been developed for estimating link (highway) travel time in the transportation system planning stage. These include the equation developed by the U.S. Bureau of Public Roads (BPR, 1964) and its modified versions (Singh, 1995; Dowling et al., 1998; AASHTO, 2003), Webster's (1958) volume-delay model, HCM (1985, 1994, and 2000) methods, Davidson's (1966, 1978) model, TRANSYT-7F models (Wallace et al., 1991 and 1998), and Akcelik's (1991) model. Among them, simplified functions that are often (practically) applied to the traffic assignment problem for estimating travel time on highway segments are the 2003 BPR (AASHTO, 2003) model and Akcelik's (1991) model. For estimating delay time on at-grade intersections and roundabouts, the HCM (2000) model is useful.

Link Performance Functions for Highway Travel Time Estimation

The BPR function has been extensively updated until recently, and currently AASHTO (2003) proposes different model parameters in terms of various road-types (e.g., freeways and arterials; refer to Table 6.4). This travel time function can characterize traffic volume-delay relationships with a simple algebraic form that is easy to remember and work with other mathematical models. The 2003 BPR model can replicate most observed patterns of delay by selecting the proper combination of free-flow travel time and the two parameters, α and β. The basic formulation of the BPR model is:

$$t_a\left(x_a\right)=t_a^f \times\left[1+\alpha\left(\frac{x_a}{c_a^{\prime}}\right)^\beta\right] \tag{6.8}$$

where: t_a = Travel time (hr/mile) predicted for traffic on arc a; $a \in \boldsymbol{A}$

$\boldsymbol{A}$ = A set of arcs in a given highway network

t_a^f = Free-flow travel time (hr/mile) on arc a; $t_a^f = L_a / V_a^f$

x_a = Traffic flow on arc a

c_a' = Practical capcity of arc a (defined as 80 % of actual capacity)

L_a = Length of arc a

V_a^f = Free-flow speed on arc a

α, β are model parameters (refer to Table 6.4)

Note that the practical capacity of arc a (denoted as c_a' in the above equation) does not equal to its actual capacity (c_a) which represents maximum possible flows that can pass through the arc in a given time; c_a' can be defined as 80% of the actual arc capacity (Dowling et al., 1998). Table 6.4 presents the BPR parameter values used in equation (6.8). In the equation, the parameter α determines the ratio of free-flow speed to the speed at capacity, and parameter β determines how abruptly the BPR curve drops from the free-flow speed.

Table 6.4 Typical BPR Function Parameters

BPR parameters	Freeway	Expressway	Arterial	Collector
Urban				
Free-flow speed (mph)	55	45	30	25
α	0.1	0.1	0.05	0.075
β	10	10	10	10
Rural/Suburban				
Free-flow speed (mph)	65	55	45	40
α	0.1	0.1	0.05	0.075
β	10	10	10	10

Source: American Association of State Highway and Transportation Officials (AASHTO): User Benefit Analysis for Highways (2003)

Akcelik (1991) proposed a modified version of Davidson's (1966 and 1978) volume-delay model for properly using it in transport planning purposes. He avoided the limitations of the steady-state form of Davidson's model by developing a different functional form that uses the free-flow travel time and queuing delay terms in an explicit way. The following relation shows the time-dependent form of Akcelik's model:

$$t_a\left(x_a\right) = t_a^f + \left\{0.25\dot{T}\left[\left(\frac{x_a}{c_a}-1\right)+\sqrt{\left(\frac{x_a}{c_a}-1\right)^2+\left(\frac{8J_A}{c_a\dot{T}}\right)\times\left(\frac{x_a}{c_a}\right)}\right]\right\} \tag{6.9}$$

where: $\dot{T}$ = Duration of traffic flow (hr); typically 1 hr

c_a= Capcity of arc a

J_A= Delay parameter (refer to Table 6.5)

Other variables are defined earlier.

Equation (6.9) implies that the travel time estimate is the sum of the free-flow travel time along the highway (the first term) and delay due to queuing (the second term); the delay is equal to the average overflow queue divided by the capacity, c_a (refer to Akcelik, 1981). Table 6.5 shows example values of the delay parameter (J_A) suggested by Akcelik (1991).

Note that although either the 2003 BPR model or Akcelik's (1991) model is applicable for predicting arc travel time in the traffic assignment process, this dissertation adopts the former travel time function because of its simpler mathematical form. A good comparison of those two models is provided in Dowling et al. (1998).

Table 6.5 Example Delay Parameters Suggested by Akcelik (1991)

Facility Type	Capacity (vphpl)	Free-Flow Speed (kph)	Free-Flow Speed (mph)	J_A	t_a^c / t_a^f
Freeway	2000	120	75	0.1	1.587
Expressway	1800	100	62	0.2	1.754
Arterial	1200	80	50	0.4	2.041
Collector	900	60	37	0.8	2.272
Local Street	600	40	25	1.6	2.439

t_a^c=Travel time on arc a when c_a=x_a

Volume-Delay Models for Intersection Delay Estimation

HCM (2000) volume-delay models (for signalized and stop-controlled intersections and roundabouts) may be employed to estimate the intersection delays in the traffic assignment process. However, because their complex functional forms (see equations (6.10) through (6.14)) may cause a significant computational burden, the models may be used only when there are critical intersections, which require detailed delay estimation, in the given highway network. Note that the volume-delay functions must be used numerous times in the traffic assignment analysis; furthermore, the assignment is processed whenever the candidate alignment is generated in the proposed network model. A simple intersection delay model, such as Webster's (1958) model may be considered for relaxing the computational complexity. However, since Webster's model does not cover oversaturated conditions[13], in which intersection demand exceeds capacity, it is unsuitable for the assignment process.

The critical intersection may be represented with several dummy nodes and arcs, as shown in Figure 6.4, and the HCM models can be applied to the dummy arcs.

[13] In Webster's formulation, the intersection-delay estimate approaches infinity when the demand equals to capacity.

For the other cases (e.g., interchanges or less important intersections connected between highway arcs), either the simple 2003 BPR model or Akcelik model can be used for the assignment process by a default. Below are the 2000 HCM intersection-delay models adopted in this dissertation:

(1) HCM volume-delay model for arcs approaching to signalized intersections:

$$d_a = d_{1\_a} \times PF + d_{2\_a} + d_{3\_a} \tag{6.10}$$

$$d_{1\_a} = 0.5Cy_{IS} \frac{\left[1 - \left(\frac{g_a}{Cy_{IS}}\right)^2\right]}{\left[1 - \min\left(1, \frac{x_a}{c_a}\right)\left(\frac{g_a}{Cy_{IS}}\right)\right]} \tag{6.11}$$

$$d_{2\_a} = 900T_d \left[\left(\frac{x_a}{c_a} - 1\right) + \left(\left(\frac{x_a}{c_a} - 1\right)^2 + \frac{8K_{IS}I_{IS}\left(\frac{x_a}{c_a}\right)}{c_a T_d}\right)^{0.5}\right] \tag{6.12}$$

where: d_a = Delay (sec/veh) for traffic on arc a approaching to an intersection

d_{1_a} = Uniform delay (sec/veh) of d_a

d_{2_a} = Incremental delay (sec/veh) of d_a

d_{3_a} = Initial queue delay (sec/veh) of d_a

PF = Progression adjustment factor; for random arrival, PF=1

Cy_{IS} = Interesection cycle length (sec)

g_a = Effective green time (sec) for traffic movement on arc a

K_{IS} = Incremental delay adjustment factor for the actuated control; for pre-timed signal, $K_{IS} = 0.5$

I_{IS} = Incremental delay adjustment factor for metering by upstream signals; for isolated intersection, $I_{IS} = 1$

Other variables are defined earlier.

The first term of equation (6.10) (i.e., equation (6.11)) represents uniform delay of traffic movement approaching a signalized intersection. The progression adjustment factor, denoted as *PF*, is set to 1.0 if arrival pattern of the traffic movement to the intersection is random. The second term of equation (6.10) (i.e., equation (6.12)) stands for incremental delay of the traffic movement that reflects non-uniform arrivals and some queue carryover between cycles within the analysis period (HCM, 2000). In the incremental delay formula there are two adjustment factors, K_{IS} and I_{IS} for the actuated control and metering by upstream signals, respectively. It is assumed that any intersection in a given highway network operates with non-actuated control (i.e., pre-timed signal) and is an isolated intersection; thus the adjustment factors for actuated control and metering are set to 0.5 and 1.0, respectively (i.e., K=0.5, I=1). Finally, the last term, the initial queue delay of equation (6.10) is assumed to be zero; i.e., it is assumed that there is no initial queue in the highway network before the traffic assignment process starts.

(2) HCM volume-delay model for arcs approaching stop controlled intersections:

$$d_a = \frac{3600}{c_a} + 900\left[\left(\frac{x_a}{c_a} - 1\right) + \left(\left(\frac{x_a}{c_a} - 1\right)^2 + \frac{\left(\frac{3600}{x_a}\right)\left(\frac{x_a}{c_a}\right)}{450}\right)^{0.5}\right] + 5 \qquad (6.13)$$

HCM (2000) defines that delay at the stop-controlled intersection is the total elapsed time from the time a vehicle stops at the end of the queue to the time the vehicle starts from the stop line. "This total elapsed time includes the time required for the vehicle to travel from the last-in-queue position to the first-in-queue position, including deceleration of vehicles from free-flow speed to the speed of vehicles in queue" (HCM, 2000).

Note that a constant value of 5 (sec/veh) is added in equation (6.13) to account for the acceleration of vehicles from the stop line to free-flow speed and the deceleration of vehicles from free-flow speed to the speed of vehicles in queue (HCM, 2000).

(3) HCM volume-delay model for arcs approaching roundabouts:

$$d_a = \frac{3600}{c_a} + 900\left[\left(\frac{x_a}{c_a} - 1\right) + \left(\left(\frac{x_a}{c_a} - 1\right)^2 + \frac{\left(\frac{3600}{x_a}\right)\left(\frac{x_a}{c_a}\right)}{450}\right)^{0.5}\right] \qquad (6.14)$$

The mathematical form of the roundabout delay model is almost the same as that of the intersection delay model for stop-controlled intersections, except for the existence of the constant term ("+5"), as shown in equations (6.13) and (6.14). According to the HCM (2000), such a difference is made to account for the fact that drivers do not need to completely stop in the approach area of the roundabout if there is no conflicting traffic.

Chapter 7: Highway Cost Formulation

This chapter describes various cost items associated with highway construction. Five major cost components incurred by highway agencies (length-dependent, right-of-way, earthwork, structure, and highway maintenance costs) are formulated in Section 7.1, and three types of user cost components (travel time, vehicle operation, and accident costs) are formulated in Section 7.2.

7.1 Agency Costs Associated with Highway Construction

In the proposed optimization model, the total agency cost (C_{T_Agency}) for new highway construction consists of (i) length-dependent cost, (ii) right-of-way (ROW) cost, (iii) earthwork costs, (iv) structure cost, and (v) highway maintenance cost. According to Jong (1998) and Kim (2001), these are dominating and alignment-sensitive costs that should be considered in the alignment optimization process. Examples associated with those cost components are summarized in Table 7.1, and the basic formulation of the total agency cost can be expressed as:

$$C_{T\_Agency} = C_L + C_R + C_E + C_S + C_M \tag{7.1}$$

where: C_{T_Agency} = Total agency cost associated with highway construction

C_L = Length-dependent cost

C_R = Right-of-way cost

C_E = Earthwork cost

C_S = Structure cost

C_M = Maintenance cost

Table 7.1 Agency Costs Associated with New Highway Construction

	Type of Agency Cost	Examples
Agency costs	Length dependent cost	Pavement[14] and median installation costs
	Right-of-way cost	Land acquisition and property damage costs
	Earthwork cost	Cut and fill costs
	Structure cost	Bridge and interchange construction costs
	Maintenance cost	Maintenance costs for highway basic segments and bridges

The mathematical formulations of the first three components (C_L, C_R, and C_E) in equation (7.1) have been well discussed in previous highway alignment optimization studies by Jong (2000), Jha (2000b), and Jha and Schonfeld (2003). This dissertation largely follows their models for evaluating such cost components of the candidate alignments.

For estimating highway structure costs (4th item in equation (7.1)), we can use Kim's (2001) models. He proposed cost models for 4-leg structures (4-leg intersections and clover and diamond interchanges), small tunnels, and bridges. Among them, 4-leg structure models are adopted in this dissertation. Note that 3-leg structure models (e.g., 3-leg intersections, trumpet interchanges, and roundabouts models) needed for representing the endpoints of the new highway and a simple bridge model are proposed in Section 5.1.

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[14] The pavement cost can be considered as length-dependent cost if width and depth of a highway are assumed to be fixed.

Besides the costs associated initial road-construction, highway maintenance costs (for highway bridges and for basic segments) should also be considered for evaluating the candidate alignments more reasonably. Simple highway maintenance cost models are proposed in Section 7.1.5. Cost functions used for estimating the five major highway-agency-costs are briefly described in the following subsections.

7.1.1 Length-Dependent Cost

The length-dependent cost (C_L) is the cost component proportional to alignment length. Initial highway pavement cost and costs required for construction of basic highway facilities for vehicle operation (such as barriers, guardrails and medians) may be included in this cost type. C_L can be expressed as:

$$C_L = K_L L_n \tag{7.2}$$

where: K_L = Unit length-dependent cost (\$/ft)

L_n = Length (ft) of a new alignment

In the optimization model, the length of a new highway alignment (L_n) is iteratively updated from the alignment generation process presented in Section 5.2. The resulting alignment consists of tangent sections and curved sections; spiral transition curves coupled with circular curves may be added to the curved sections. The alignment length can be expressed as follows:

Case 1: for horizontal circular curves with spiral transitions (see Section 5.2)

$$L_n = \sum_{i=0}^{n} \sqrt{\left(x_{ST_i} - x_{TS_{i+1}}\right)^2 + \left(y_{ST_i} - y_{TS_{i+1}}\right)^2} + \sum_{i=1}^{n} \left(l_{ST_{i1}} + l_{ST_{i2}} + R_{C_i}(\theta_{PI_i} - 2\theta_{ST_i}\right) \tag{7.3a}$$

where: (x_{TS_i}, y_{TS_i}) = Coordinates of $\mathbf{TS}_i$, which are given in equation (5.39)

(x_{ST_i}, y_{ST_i}) = Coordinates of $\mathbf{ST}_i$, which are given in equation (5.40)

$l_{ST_{i1}}, l_{ST_{i2}}$ = Length of spiral transition curves at both ends of i_{th} circular curve; assume $l_{S_{i1}} = l_{S_{i2}}$; refer to equation (5.37)

R_{C_i} = Radius of i_{th} circular curve

$\theta_{PI\,i}$ = Deflection angle at $\mathbf{PI}_i$, which is given in equation (5.38)

θ_{ST_i} = Spiral angle at $\mathbf{PI}_i$; refer to Table 5.2

Case 2: for horizontal circular curves without spiral transitions (refer to Jong, 1998)

$$L_n = \sum_{i=0}^{n} \sqrt{\left(x_{T_i} - x_{C_{i+1}}\right)^2 + \left(y_{T_i} - y_{C_{i+1}}\right)^2} + \sum_{i=1}^{n} R_{C_i}\theta_{PI_i} \tag{7.3b}$$

where: (x_{TC_i}, y_{TC_i}) = Coordinates at the beginning of i_{th} circular curve

(x_{CT_i}, y_{CT_i}) = Coordinates at the end of i_{th} circular curve

Equation (7.3a) is used for calculating the alignment length when one circular and two spiral transition curves are incorporated in a horizontal curve, while equation (7.3b) is employed when only a circular curve is used.

7.1.2 Right-of-Way (ROW) Cost

The right-of-way cost (C_R) of a new highway is not simply a sum of the land values taken by the alignment. The reduction in value due to a nearby alignment, as well as the usability of the remaining land, should also be considered. With such considerations, Jha (2000) and Jha and Schonfeld (2000b) formulated detailed right-of-way cost using the GIS application. They specified three sub items of the right-of-way cost: (i) temporary easement costs, (ii) just compensation costs, and (iii) appraisal fees. The temporary easement costs are defined as the partial taking of a property during the construction. The just compensation costs represents damage, site improvements, and cost of the fraction of property affected by the alignment itself. The right-of-way cost function adopted in this dissertation is:

$$C_R = \sum_{i=1}^{n_R} \left(C_{TE_i} + C_{JC_i} + C_{AF_i} \right) \tag{7.4}$$

where: C_{TE_i} = Cost of 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

n_R = Total number of properties affected by the alignment, and

$$C_{JC_i} = C_{DP_i} + C_{DS_i} + C_{SI_i} + C_{F_i} \tag{7.5}$$

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

C_{F_i} = Cost of the fraction of property i affected by the alignment

Detailed descriptions of the above major cost components comprising the right-of-way costs may be found in earlier publications (especially, Jha 2000; Jha and Schonfeld, 2000b).

7.1.3 Earthwork Cost

Another major component of the agency cost is the cost (C_E) required for earthwork. Jha and Schonfeld (2003) proposed a detailed earthwork-cost model based on the average end area method[15]. Their earthwork-cost model is:

$$C_E = C_H + \sum_{i=1}^{n_r} \left[\begin{array}{l} \left(\dfrac{\omega_1 K_{ci} s_r A_{ci} L_i}{2} \right) + \left(\dfrac{\omega_2 K_{fi} A_{fi} L_i}{2} \right) + \\ \left((1 - \omega_1 - \omega_2) L_i \left(\dfrac{K_{ci} s_r A_{tci} + K_{fi} A_{tfi}}{2} \right) \right) \end{array} \right] \tag{7.6}$$

where: C_H = Total haul cost; C_H is a function of unit hauling cost and haul distance, and is provided in Jha (2000)

n_r = Total number of cut and fill sections

L = Alignment length either at a cut, fill, or transition section

s_r = Earth shrinkage or swell factor (decimal)

K_c and K_f = Unit cut and fill costs

A_c and A_f = The end areas for a cut section and for a fill section

A_{tc} and A_{tf} = The cut and fill areas in a transition section

$$\begin{cases} \omega_1 = 1,\ \omega_2 = 0 & \text{in a cut section} \\ \omega_1 = 0,\ \omega_2 = 1 & \text{in a fill section} \\ \omega_1 = \omega_2 = 0 & \text{in a transition section} \end{cases}$$

The data required for earthwork volume computation are (i) terrain profile of the study area (i.e., ground elevation) and (ii) road heights at each major break in the terrain surface along the vertical alignment. The study-area ground elevation is obtainable from an input GIS data (e.g., DEM), and the vertical alignment is

[15] The average end area method is the one most commonly used in computing the earthwork volume. This method is based on the assumption that the volume between two successive cross sections is the average of their areas multiplied by the distance between them.

generated from the *Road Elevation Determination Procedure (3.2)*. The unit-cut and unit-fill costs and earth shrinkage factor are user-specifiable.

A detailed description of equation (7.6) may be found in Jha and Schonfeld (2003), and the mathematical formulation of C_H is provided in Jha (2000).

7.1.4 Highway Structure Cost

Many highway structures are also associated with the construction of a new highway. These may include bridges for crossing the rivers or valleys and cross-structures for intersecting existing highways. Since the costs required for those structures are sensitive and dominating in highway construction, they should also be included in the total agency cost. In alignment optimization, a basic model for estimating the highway structure cost (C_S) can be expressed as:

$$C_S = C_{BR} + C_{IC} + C_{IS} + C_{GS} \qquad (7.7)$$

where: C_{BR} = Bridge construction cost

C_{IC} = Interchange construction cost

C_{IS} = Intersection construction cost

C_{GS} = Grade separation cost (for over and underpasses)

For estimating cost of a small highway bridge, which is mostly used for grade separation of existing roads, Kyte et al.'s (2003) simple regression model is useful coupled with findings from Menn's (1990) study. For estimating construction cost of a bridge designed for crossing rivers or valleys, O'Connor's (1971) theoretical model can be used. Note that those bridge models are discussed in Section 5.1.2.2.

Besides bridge structures, three types of cross-structures (for intersecting existing highways) are considered in the alignment optimization process. These are interchanges, intersections, and grade-separation structures. For estimating construction costs of 4-leg cross-structures (such as, clover, diamond, and 4-leg intersections), Kim's (2001) models are usable. However, we still need to model 3-leg cross-structures (e.g., 3-leg intersections, trumpet interchanges, and roundabouts) which are basically considered at the points of existing highways from which a new highway (or a bypass) extends to other directions. Simple models for estimating construction costs of the 3-leg structures are presented in Section 5.1.2. For simplification in the model formulation, the cost models are developed based on the centerline drawings of those structures.

7.1.5 Highway Maintenance Cost

Besides the above cost components that are initially required for construction of a new highway (e.g., ROW, earthwork, and structure costs), this dissertation also considers the highway maintenance cost. We subdivide the total highway facility into two sub-categories - (i) highway basic segments and (ii) highway bridges - since different sources of unit costs are used for estimating their maintenance costs in literature. A basic model for the total highway maintenance cost (C_M) can be expressed as follows:

$$C_M = C_{HM} + C_{BO} \tag{7.8}$$

where: C_{HM} = Present value of maintenance cost for highway basic segments

C_{BO} = Present value of bridge operating cost

(1) Maintenance Cost for Highway Basic Segments

The highway maintenance cost (C_{HM}) is normally length-dependent; that is, it is proportional to the length of the road segment. Thus, given with the length of the highway segment and its unit maintenance cost (normally \$ per unit distance per year), the highway maintenance cost can be estimated as follows:

$$
\begin{aligned}
C_{HM} &= \left(L_n - \sum_{i=1}^{n_{BG}} l_{BG_i} \right) \left(K_{AM} \sum_{k=1}^{n_y} \left(\frac{1}{(1+\rho)} \right)^k \right) \\
&= \left(L_n - \sum_{i=1}^{n_{BG}} l_{BG_i} \right) \left(K_{AM} \left(\frac{1 - e^{(-\rho) n_y}}{\rho} \right) \right)
\end{aligned}
\qquad (7.9)
$$

where: L_n = Length of a new alignment, which is given in equation (7.3)

l_{BG} = Bridge length, which is given in equation (5.33)

n_{BG} = Number of highway bridges

K_{AM} = Annual average maintenance cost per unit length (\$/ft/yr)

ρ = Assumed interest rate (decimal fraction)

n_y = Analysis period (yrs)

In equation (7.9), K_{AM} normally includes costs of routine highway maintenance required annually, such as repair of roadway pavement, guardrail, and median and drainage. Road resurfacing and rehabilitation costs may be included in the maintenance cost if the project evaluation period exceeds the highway's design-life. The value of K_{AM} can be found from many studies on the subject. For instance, Safronetz and Sparks (2003) used \$0.888/ft/yr (\$2,915/km/yr) for estimating the road maintenance cost in their highway management model, and Christian and Newton (1999) proposed \$0.914/ft/yr (\$3,000 /km/yr) for the unit highway maintenance cost.

(2) Operating Cost for Highway Bridges

Bridge operating cost (C_{BO}) is incurred as a result of annual inspection, annual maintenance, and periodic rehabilitation. Menn (1990) provided yearly operating cost (C_{AB}) of a highway bridge as a percentage (K_{AB}) of its initial construction cost (C_{BR}). As shown in Table 7.2, 1 to 1.2 % of the initial bridge construction cost is spent for bridge operation annually, on average. Thus, if any bridge construction cost is available, its annual bridge operating cost could be roughly estimated. The bridge operating cost can be estimated with the following formula:

$$C_{BO} = \sum_{i=1}^{n_{BG}} \left[C_{AB^i} \sum_{k=1}^{n_y} \left(\frac{1}{(1+\rho)} \right)^k \right] = \sum_{i=1}^{n_{BG}} \left[\left(\frac{1}{100} K_{AB} C_{BR^i} \right) \left(\frac{1-e^{(-\rho)n_y}}{\rho} \right) \right] \qquad (7.10)$$

where: C_{AB} = Annual bridge operation cost (\$/yr); $C_{AB} = \frac{1}{100} K_{AB} C_{BR}$

C_{BR} = Bridge construction cost

K_{AB} = A percentage of the bridge construction cost (C_{BR}) for C_{AB}; see Table 7.2 for its value

n_{BG} = Number of highway bridges

Table 7.2 Annual Bridge Operating Cost (average over the bridge lifetime)

Item	% of bridge construction cost (K_{AB})
Inspection	0.1
Maintenance	0.5
Rehabilitation	0.4 to 0.6
Total	1 to 1.2

Source: Menn (1990)

7.2 User Cost Savings from New Highway Addition

The drivers' route choice behavior can change before and after a new highway addition to the network, and may vary for its different alternatives. To consider such variation in the model evaluation process, this dissertation considers the user cost components of the different alternatives in the model objective function besides their agency costs presented in the previous section.

Let C^0_{User} and C^1_{User} be total network user costs before and after a new highway construction, respectively. Then, we may expect either positive or negative user cost savings (denoted as $\Delta C_{User} = C^0_{User} - C^1_{User}$) from the road construction. The network users may save their travel time, fuel consumption, and accident costs through the addition of a good highway alternative; however, they may pay more such cost items due to the addition of undesirable one. Three types user cost saving considered in the model are presented in Table 7.3.

Table 7.3 Type of User Cost Savings

	Type of Cost	Examples
User-cost savings	Travel time cost saving	Decrease or increase in travel time
	Vehicle operating cost saving	Decrease or increase in fuel consumption
	Accident cost saving	Decrease or increase in no. of accidents

Recall that we assume the overall origin-destination flows in a small highway network are stable with and without a new highway addition (i.e., $q^0_{rs} = q^1_{rs}\ \forall r, s\ (r \neq s)$ where q^0_{rs} and q^1_{rs} are trip rates between origin r and destination s before and after the new highway construction, respectively; $Q^0 = \sum q_{rs}^0$ and $Q^1 = \sum q_{rs}^1 \; \forall r, s \; (r \neq s)$); however, individual network users can freely select their travel paths. Given with the constant overall demand assumption, the following relation describes a basic formulation used to evaluate the user cost savings (ΔC_{User}).

$$\begin{aligned}
\Delta C_{User} &= C_{User}^0 - C_{User}^1 = \left(C_T^0 + C_V^0 + C_A^0\right) - \left(C_T^1 + C_V^1 + C_A^1\right) \\
&= \left(C_T^0 - C_T^1\right) + \left(C_V^0 - C_V^1\right) + \left(C_A^0 - C_A^1\right) \\
&= \Delta C_T + \Delta C_V + \Delta C_A
\end{aligned} \tag{7.11}$$

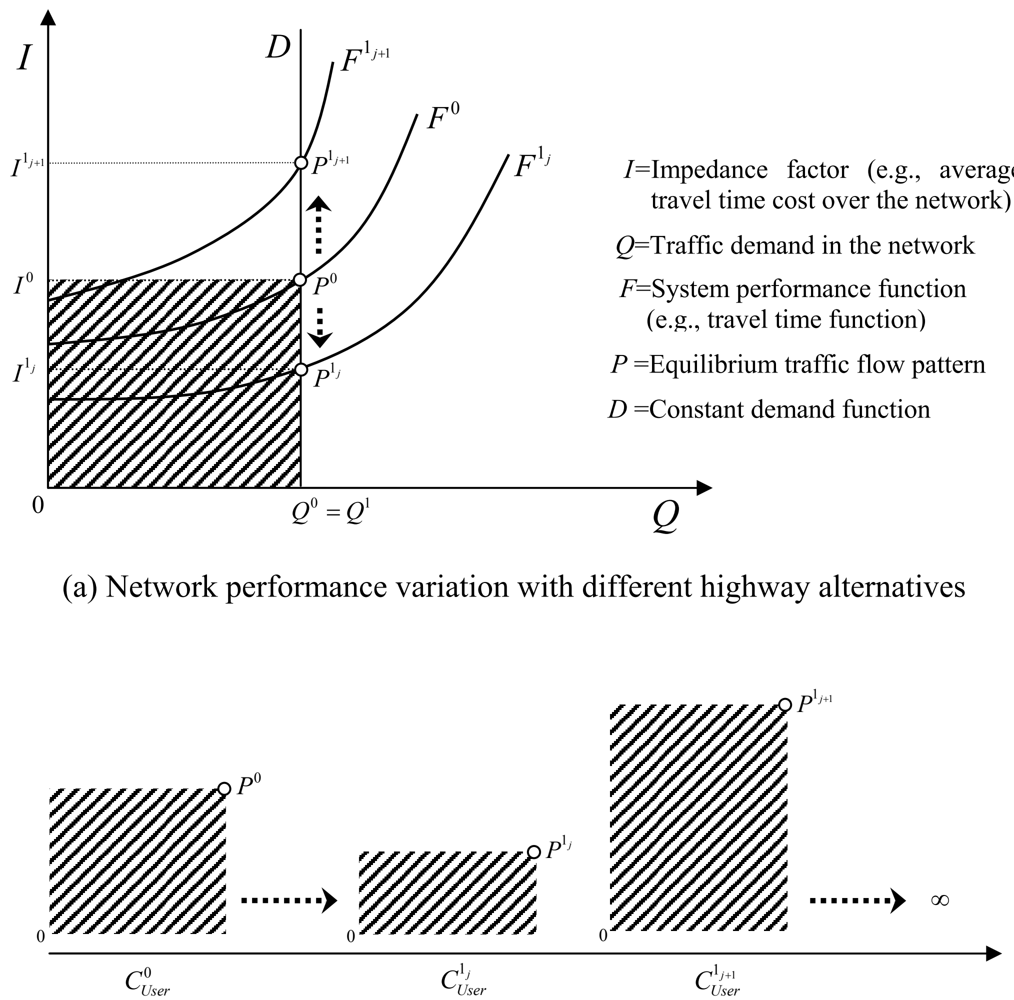
where: C_{User}^0, C_{User}^1 = Total user cost before and after the new highway-construction, respectively

C_T^0, C_V^0, C_A^0 = Travel time cost, vehicle operating cost, and accident-cost over the network without highway construction, respectively

C_T^1, C_V^1, C_A^1 = Travel time cost, vehicle operating cost, and accident-cost over the network after the new highway-construction, respectively

$\Delta C_T, \Delta C_V, \Delta C_A$ =Expected savings in travel time cost, vehicle-operating cost, and accident cost, respectively after the new highway construction

A basic economic concept used for evaluating the user cost savings is presented in Figure 7.1. In its notation, superscripts *0* and *1* stand for conditions of a given highway network before and after a new alignment addition. For example, F^{1_j} represents a conceptual traffic performance function (e.g., travel time function) of the network after addition of a highway alternative *j*. In the model, numerous highway alternatives (*j*=1 to ∞) can be generated from the alignment generation procedure during the optimization process, and equilibrium traffic flow patterns over the network by the addition of the different alternatives are automatically updated from the traffic assignment process.



(a) Network performance variation with different highway alternatives

(b) Total user-cost variation with different highway alternatives

Figure 7.1 A Basic Economic Concept for Evaluating User Cost Savings

Based on the concept used in equation (7.11) and Figure 7.1, the three user cost components for different highway alternatives are evaluated in the optimization process (see the following sub-sections). The cost evaluation procedure presented in each sub-section modifies and adjusts the basic steps of the user benefit analysis proposed in AASHTO (2003) for major highway construction projects. Additionally, since traffic patterns vary with time of day, different time periods during a day (AM, PM, and OFF peaks) are considered for more precise cost estimation. According to the AASHTO (2003), a model day of 18 hours is suggested, with traffic in six hours from 12 midnight to 6 AM added to the off-peak period; thus, there are 309 weekdays in a year. We assume the total number of AM and PM peak hours in a year are $309H_{AM}$ and $309H_{PM}$, respectively, and total number of off-peak hours in year is $(365\times18)-309\times(H_{AM}+H_{PM})$.

7.2.1 Travel Time Cost

The travel time cost is calculated based on the amount of time spent for traveling and the drivers' perceived value of time. Before elaborating on the travel time cost model, it is important to note that there are multiple user classes operating on the highway network, and each user class may have different trip purposes[16]. In this dissertation, it is assumed that two types of user classes (auto and truck) operate on the highway network, and they have different values of travel time with respect to different trip purposes (see Table 7.4). A more detailed trip purpose factor for each user class (such as, home-based work, home-based other, and non-home-based trips)

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[16] The economic valuation of the travel time cost generally varies with different user classes and with different trip purposes.

is not considered here, since it may not be available in the initial stage of the highway project due to either time or money constraint (or both).

Table 7.4 shows wage compensation rates suggested in AASHTO (2003) with respect to different trip purposes. With these guidelines, the unit travel time value for each user class can be simply calculated by multiplying the wage compensation rate (here we use average values shaded in Table 7.4) by the corresponding average wage shown in Table 7.5.

Table 7.4 Wage Compensation Rate for Different Trip Purposes

Mode	Trip Purpose	Percentage of wage compensation
Auto	Drive alone commute	50% of the wage rate
	Carpool driver commute	60% of the wage rate
	Carpool passenger commute	40% of the wage rate
	Personal	50%~70% of the wage rate
	Average	**50% of the wage rate**
Truck	**In-vehicle and excess (waiting time) business**	**100% of total compensation**

Source: AASHTO and U.S. Department of Transportation. 1997. The Value of Travel Time: Departmental Guidance for Conducting Economic Evaluations. Washington, D.C.

Table 7.5 Average Wages, by Industry (2000 U.S dollars)

Industry Type	Average Wage ($/hr)
All employees (Auto users)	$18.56
Truck drivers	$16.84

Source: National Income and Product Accounts (NIPA) of the United States, for 2000 (Department of Commerce, Bureau of Economic Analysis).

Let **v** be the vector of unit travel time values for auto and truck drivers. Then, the values can be estimated as:

$$\mathbf{v} = \begin{bmatrix} v_{Auto} \\ v_{Truck} \end{bmatrix} = \begin{bmatrix} 0.5 \times 18.56 \\ 1.0 \times 16.84 \end{bmatrix} = \begin{bmatrix} 9.28 \\ 16.84 \end{bmatrix} \tag{7.12}$$

where: v_{Auto} = Unit travel time value (\$/hr) for auto drivers
v_{Truck} = Unit travel time value (\$/hr) for truck drivers

Vehicle occupancy information is also required to evaluate travel time cost of highway users more precisely. Let **o** be the vector of the average vehicle occupancy for the auto and truck drivers in the traffic flows. Table 7.6 presents average vehicle occupancy information from the National Personal Travel Survey (NPTS, 1995). In vector representation **o** can be expressed as:

$$\mathbf{o} = \begin{bmatrix} o_{Auto} \\ o_{Truck} \end{bmatrix} = \begin{bmatrix} 1.550 \\ 1.144 \end{bmatrix} \tag{7.13}$$

where: o_{Auto} = Average vehicle occupancy for auto drivers
o_{Truck} = Average vehicle occupancy for truck drivers

Table 7.6 Average Vehicle Occupancy for Auto and Truck

Vehicle Types	Average Vehicle Occupancy
Auto	1.550 (persons/vehicle)
Truck	1.144 (persons/vehicle)

Source: Oak Ridge National Laboratories, *1995 National Personal Travel Survey*, Table NPTS-1, October 1997 (www.cta.ornl.gov/npts/1995/doc/tabel1.pdf)

Additionally, we define a traffic composition vector (denoted as **T**) for autos and trucks in the traffic flows. The truck percentage (*T*) is needed as a model input (so the auto percentage in the traffic= 1-*T*), and **T** can be expressed as:

$$\mathbf{T} = \begin{bmatrix} 1-T \\ T \end{bmatrix} \tag{7.14}$$

where: T = The proportion (decimal fraction) of trucks in traffic flows over the network.

We now estimate economic value of the travel-time cost savings based on (i) the values obtained from equations (7.12) through (7.14) and (ii) traffic performance measures obtained from the traffic assignment process (for before and after the new highway construction). The following steps show how the travel-time saving is estimated in the optimization process:

STEP 1: Update traffic volume and travel time on all highways in the network from the traffic assignment process

$$\left(\begin{array}{l} \text{for } AM \text{ peak } : x_{a^0}^{AM},\, t_{a^0}^{AM},\, x_{a^1}^{AM},\, t_{a^1}^{AM} \\ \text{for } PM \text{ peak } : x_{a^0}^{PM},\, t_{a^0}^{PM},\, x_{a^1}^{PM},\, t_{a^1}^{PM} \\ \text{for } OFF \text{ peak} : x_{a^0}^{OFF},\, t_{a^0}^{OFF},\, x_{a^1}^{OFF},\, t_{a^1}^{OFF} \end{array} \right. \qquad \forall a^0 \in \boldsymbol{A}^0 \text{ and } \forall a^1 \in \boldsymbol{A}^1$$

where: $\boldsymbol{A}^0$ = A set of arcs in the existing road network; $a^0 \in \boldsymbol{A}^0$

$\boldsymbol{A}^1$ = A set of arcs in the network updated after a new highway-construction; $a^1 \in \boldsymbol{A}^1$

$x_{a^1}^{AM}$, $x_{a^1}^{PM}$, $x_{a^1}^{OFF}$ are average traffic flows (vph) on arc a^1 during AM, PM, and OFF peak hours, respectively after a new highway construction.

$x_{a^0}^{AM}$, $x_{a^0}^{PM}$, $x_{a^0}^{OFF}$ are average traffic flows (vph) on arc a^0 during AM, PM, and OFF peak hours, respectively, without highway construction.

$t_{a^1}^{AM}$, $t_{a^1}^{PM}$, $t_{a^1}^{OFF}$ are average travel times (hr) on arc a^1 during AM, PM, and OFF peak hours, respectively after a new highway construction.

$t_{a^0}^{AM}$, $t_{a^0}^{PM}$, $t_{a^0}^{OFF}$ are average travel times (hr) on arc a^0 during AM, PM, and OFF peak hours, respectively without highway construction.

Note that $x_{a^0}^{AM}$, $t_{a^0}^{AM}$, $x_{a^0}^{PM}$, $t_{a^0}^{PM}$, $x_{a^0}^{OFF}$, and $t_{a^0}^{OFF}$ are computed only once at the beginning of the optimization process.

STEP 2: Calculate total travel time cost over the network

$$C_{T\_B}^{1} = \sum_{a^1 \in \boldsymbol{A}^1} \left(\begin{bmatrix} x_{a^1}^{AM} \\ x_{a^1}^{PM} \\ x_{a^1}^{OFF} \end{bmatrix} \cdot \begin{bmatrix} t_{a^1}^{AM} \\ t_{a^1}^{PM} \\ t_{a^1}^{OFF} \end{bmatrix} \cdot \begin{bmatrix} H_{TAM} \\ H_{TPM} \\ H_{TOFF} \end{bmatrix} \right) [\mathbf{v} \cdot \mathbf{T} \cdot \mathbf{o}] \tag{7.15a}$$

$$C_{T\_B}^{0} = \sum_{a^0 \in \boldsymbol{A}^0} \left(\begin{bmatrix} x_{a^0}^{AM} \\ x_{a^0}^{PM} \\ x_{a^0}^{OFF} \end{bmatrix} \cdot \begin{bmatrix} t_{a^0}^{AM} \\ t_{a^0}^{PM} \\ t_{a^0}^{OFF} \end{bmatrix} \cdot \begin{bmatrix} H_{TAM} \\ H_{TPM} \\ H_{TOFF} \end{bmatrix} \right) [\mathbf{v} \cdot \mathbf{T} \cdot \mathbf{o}] \tag{7.15b}$$

where: $C_{T\_B}^{1}$ = Total travel time cost (\$/yr) over the network after a new highway-construction in the base year

$C_{T\_B}^{0}$ = Total travel time cost (\$/yr) over the network without highway-construction in the base year

H_{TAM} = Total number of AM peak hours per year; $H_{TAM} = 309 H_{AM}$

where, $H_{AM} = AM$ peak duration (hrs) per day

H_{TPM} = Total number of PM peak hours per year; $H_{TPM} = 309 H_{PM}$

where, $H_{PM} = PM$ peak duration (hrs) per day

H_{TOFF} = Total number of OFF peak hours per year;

$= 365 \times 18 - (H_{TAM} + H_{TPM})$

$\cdot$ = Inner(dot) product

STEP 3: Calculate present value of total travel time cost saving

$$\Delta C_T = \left(C_{T\_B}^{0} - C_{T\_B}^{1} \right) \times \left[\frac{e^{(r_t - \rho) n_y} - 1}{r_t - \rho} \right] \tag{7.16}$$

where: ΔC_T = Present value of total travel time cost saving after a new-highway construction for the analysis period n_y

ρ = Assumed interest rate (decimal fraction)

r_t = Annual growth rate of traffic over the network (decimal fraction)

n_y = Analysis period

Note that intersections may entail additional travel time (delay) besides the arc travel time (t_a), while providing right-of ways to all turning movements entering the intersection. Thus, if there is any intersection in the highway network, the intersection delay (d_a) may also be considered in the travel time cost estimation procedure presented above. Intersection-delay functions used in the model are presented in Section 6.3.2.

7.2.2 Vehicle Operating Cost

Another user cost component considered in the model is the 'vehicle operating cost' that can be directly perceived by drivers (the network users) as an out-of-pocket expense incurred while operating vehicles. This may include fuel and oil, maintenance, tire wear, and vehicle depreciation costs. However, since the vehicle depreciation cost is not sensitive to network configuration with different highway alternatives to be added, only fuel consumption and vehicle maintenance (including tire wear cost) costs, which are the most dominating and sensitive ones, are considered in this analysis. Generally, the vehicle operating cost can be calculated on a per vehicle-mile basis; link distance as well as equilibrium link traffic information (such as flow, travel time, and speed on each link), which are obtained from the traffic assignment process, are used in order for estimating the vehicle operating cost. The fuel consumption (efficiency) indices, expressed in gallons per mile at different average operation speeds, for the two different user-classes (auto and truck) are presented in Table 7.7, and their fuel prices (dollars per gallon) and average maintenance and tire costs are shown in Table 7.8. These are also required inputs for estimating the vehicle operation cost.

Table 7.7 Fuel Consumption Rates for Auto and Truck

Speed	Gallons/mile	
	Auto	Truck
5 mph	0.117	0.503
10 mph	0.075	0.316
15 mph	0.061	0.254
20 mph	0.054	0.222
25 mph	0.050	0.204
30 mph	0.047	0.191
35 mph	0.045	0.182
40 mph	0.044	0.176
45 mph	0.042	0.170
50 mph	0.041	0.166
55 mph	0.041	0.163
60 mph	0.040	0.160
65 mph	0.039	0.158

Source: Inputs to SPASM Based on Cohn, et al., 1992. "Environmental and Energy Considerations," in *Transportation Planning Handbook.* Inst. of Transportation Engineers.

Table 7.8 Auto and Truck Fuel Prices and Maintenance and Tire Costs

Category	Auto	Truck
Fuel	2.73 dollars/gallon	2.67 dollars/gallon
Maintenance and Tires*	0.04 dollars/mile	0.05 dollars/mile

* Source: American Automobile Association and Runzheimer International, *Your Driving Costs*, 1999 Edition. Data for a popular model of each type listed with ownership costs based on 60,000 miles before replacement. Adjusted to 2002 dollars by ECONorthwest.

Then, the unit vehicle-operating cost can be estimated with the sum of (i) unit fuel consumption costs, calculated by multiplying the fuel consumption and the fuel price, and (ii) maintenance and tire costs. The unit vehicle-operating cost can be expressed as:

$$\mathbf{f}_a = \begin{bmatrix} p_{Auto} f_{a\_Auto} \\ p_{Truck} f_{a\_Truck} \end{bmatrix} + \begin{bmatrix} m_{Auto} \\ m_{Truck} \end{bmatrix} = \begin{bmatrix} 2.73\left(0.194\bar{V}_{a\_Auto}^{-0.4009}\right) + 0.04 \\ 2.67\left(0.869\bar{V}_{a\_Truck}^{-0.4286}\right) + 0.05 \end{bmatrix} \tag{7.17}$$

where: $\mathbf{f}_a$ = Vector representation of unit vehilce-operating cost (\$/mile) for traffic flow operating on arc a

$\boldsymbol{A}$ = A set of arcs in a given highway network; $a \in \boldsymbol{A}$

p_{Auto}, p_{Truck} are fuel prices (\$/gallon) of autos and trucks, respectively.

f_{a_Auto}, f_{a_Truck} are fuel efficiencies (gallons/mile) for autos and trucks, operating on arc a with average speeds $\bar{V}_{a\_Auto}$ and $\bar{V}_{a\_Truck}$, respectively; normally $\bar{V}_{a\_Auto} = \bar{V}_{a\_Truck}$.

m_{Auto}, m_{Truck} are vehicle maintenance and tire wear costs (\$/mile) for autos and trucks, respectively.

As shown in equation (7.17), we use \$2.73 and \$2.67 per gallon for fuel prices of auto and truck modes, respectively, and \$0.040 and \$0.050 per mile are used for representing their unit vehicle maintenance and tire wear costs (see Table 7.8). Since the fuel efficiency varies depending on different modes (here autos and trucks) as well as their operating speeds, we use regression functions obtained from the data provided in Table 7.7. Figure 7.2 shows unit fuel-consumption functions for the two different modes resulting from the regression analysis.

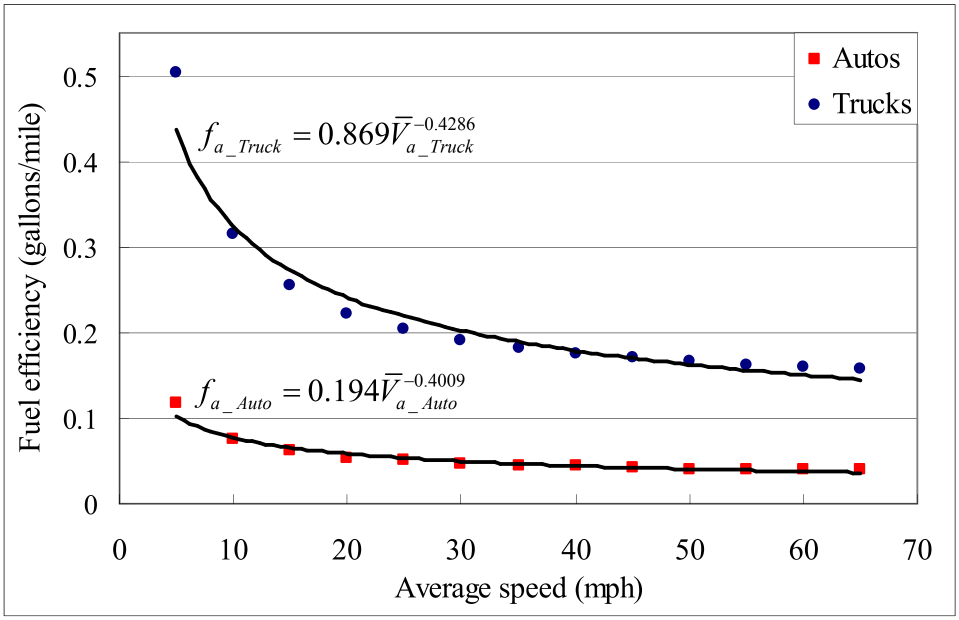

Figure 7.2 Unit Fuel Consumption Functions for Auto and Truck

The total vehicle operation cost for users travels on the network can be calculated by multiplying the unit value of the vehicle operating cost (given per vehicle-mile for corresponding link) by the arc travel distance and corresponding link traffic volume. Note that the total vehicle operating cost in the given highway network may increase after the new highway construction since the construction scenario entails more links (road segments), and thus possibly more overall travel in the study area. However, "the average vehicle operating cost per vehicle-mile over the study area would be expected to decrease with the improved network; in other words, under improved travel conditions, the per-unit cost of travel decreases" (Clifton and Mahmassani, 2004). The following steps show how the vehicle operating cost saving is estimated in our optimization model:

STEP 1: Update alignment length, traffic volume, and unit vehicle operating cost of all highways from the traffic assignment process

$$L_{a^0},\ L_{a^1},\ \text{and} \left(\begin{array}{l} \text{for } AM \text{ peak } : x_{a^0}^{AM},\ \mathbf{f}_{a^0}^{AM},\ x_{a^1}^{AM},\ \mathbf{f}_{a^1}^{AM} \\ \text{for } PM \text{ peak } : x_{a^0}^{PM},\ \mathbf{f}_{a^0}^{PM},\ x_{a^1}^{PM},\ \mathbf{f}_{a^1}^{PM} \\ \text{for } OFF \text{ peak } : x_{a^0}^{OFF},\ \mathbf{f}_{a^0}^{OFF},\ x_{a^1}^{OFF},\ \mathbf{f}_{a^1}^{OFF} \end{array} \right. \qquad \forall a^0 \in \boldsymbol{A}^0 \text{ and } \forall a^1 \in \boldsymbol{A}^1$$

where: L_{a^0}, L_{a^1} length of arc a^0 and a^1, respectively.

$\mathbf{f}_{a^1}^{AM}$, $\mathbf{f}_{a^1}^{PM}$, $\mathbf{f}_{a^1}^{OFF}$ are vector representations of unit vehicle operating-cost ($/mile) for traffic (autos and trucks) on arc a^1 during AM, PM, and OFF peak hours, respectively after a new highway construction.

$\mathbf{f}_{a^0}^{AM}$, $\mathbf{f}_{a^0}^{PM}$, $\mathbf{f}_{a^0}^{OFF}$ are vector representations of unit vehilce operating-cost ($/mile) for traffic (autos and trucks) on arc a^0 during AM, PM, and OFF peak hours, respectively, without highway construction; refer to equation (7.17).

Note that $x_{a^0}^{AM}, x_{a^0}^{PM}, x_{a^0}^{OFF}, \mathbf{f}_{a^0}^{AM}, \mathbf{f}_{a^0}^{PM}$, and $\mathbf{f}_{a^0}^{OFF}$ are computed only once at the beginning of the optimization process.

STEP 2: Calculate total vehicle operating cost over the network

$$C_{V\_B}^1 = \frac{1}{5280} \sum_{a^1 \in \boldsymbol{A}^1} L_{a^1} \left(\begin{bmatrix} x_{a^1}^{AM} \\ x_{a^1}^{PM} \\ x_{a^1}^{OFF} \end{bmatrix} \cdot \begin{bmatrix} \mathbf{f}_{a^1}^{AM} \cdot \mathbf{T} \\ \mathbf{f}_{a^1}^{PM} \cdot \mathbf{T} \\ \mathbf{f}_{a^1}^{OFF} \cdot \mathbf{T} \end{bmatrix} \cdot \begin{bmatrix} H_{TAM} \\ H_{TPM} \\ H_{TOFF} \end{bmatrix} \right) \tag{7.18a}$$

$$C_{V\_B}^0 = \frac{1}{5280} \sum_{a^0 \in \boldsymbol{A}^0} L_{a^0} \left(\begin{bmatrix} x_{a^0}^{AM} \\ x_{a^0}^{PM} \\ x_{a^0}^{OFF} \end{bmatrix} \cdot \begin{bmatrix} \mathbf{f}_{a^0}^{AM} \cdot \mathbf{T} \\ \mathbf{f}_{a^0}^{PM} \cdot \mathbf{T} \\ \mathbf{f}_{a^0}^{OFF} \cdot \mathbf{T} \end{bmatrix} \cdot \begin{bmatrix} H_{TAM} \\ H_{TPM} \\ H_{TOFF} \end{bmatrix} \right) \tag{7.18b}$$

where: $C_{V\_B}^1$ = Total vehicle operating cost ($/yr) over the network after a new highway construction in the base year

$C_{V\_B}^0$ = Total vehicle operating cost ($/yr) over the network without highway construction in the base year

$\mathbf{T}$ = Traffic composition vector for autos and trucks; see equation (7.14)

$\cdot$ = Inner(dot) product

STEP 3: Calculate present value of total vehicle operating cost saving

$$\Delta C_V = \left(C_{V\_B}^0 - C_{V\_B}^1\right) \times \left[\frac{e^{(r_t-\rho)n_y} - 1}{r_t - \rho}\right] \tag{7.19}$$

where: ΔC_V = Present value of total vehicle operating cost saving after a new highway construction for the analysis period n_y, and other parameters are as defined earlier.

An at-grade intersection may also increase vehicle fuel cost to the highway users since it delays and stops vehicles. Thus, if any at-grade intersections are included in the given highway network, the vehicle fuel cost associated with the intersections should also be included in the total vehicle operating cost estimated for the network. The estimation of intersection vehicle operating cost (both for signalized and un-signalized intersections) is sufficiently discussed in Kim (2001)[17], so it is not repeated here.

7.2.3 Accident Cost

Estimating highway accident cost is relatively difficult since accidents are caused by combinations of various factors (such as traffic volume, highway geometry, and driving conditions of users operating on a highway).

Generally, for estimating the accident cost, it is necessary to determine two distinct elements: (i) *accident frequency* (i.e., the number of accidents) and (ii) *accident unit cost* ($/accident). The *accident frequency* reflects the likelihood of an accident occurring on a given highway segment or feature (e.g., an intersection), and

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[17] Kim (2001) used Webster (1958) and HCM (2001) models to calculate the intersection fuel cost, and estimated intersection vehicle operating cost based on the fuel cost.

is normally predicted with a regression analysis based on historic accident data. Some useful accident prediction model for predicting number of accidents on highway segments and intersections are discussed in the next sub-sections. The *accident unit cost* (denoted as U_{ACC} in this paper) represents cost of traffic accidents (including property damage, injury, and death) perceived by the highway users. Table 7.9 summarizes the *accident unit cost* value provided by AASHTO (2003); note that according to the AASHTO (2003), normally net-perceived-user-cost ($53,900/accident in the table) is used for U_{ACC}.

Table 7.9 Accident Unit Cost (year 2000 dollar/accident)

Accident Type	Average Perceived User Cost	Average Insurance Reimbursement	Net Perceived User Cost
All Accidents (including fatal, injury, and property damages)	69,300	15,400	53,900

Source: U.S. DOT, National Highway Traffic Safety Administration, *The Economic Impact of Motor Vehicle Crashes 2000*, U.S. DOT, FHWA, *Technical Advisory on Motor Vehicle Accident Costs*, 1994 (values converted to 2000 dollars by ECONorthwest); U.S. DOT, National Highway Traffic Safety Administration, *Traffic Safety Facts 2000*, December 2001; Insurance Research Council, *Trends in Auto Injury Claims, 2000 Edition*, 2001

7.2.3.1 Accident Cost for Basic Highway Segments

A variety of accident prediction models have been developed for predicting accidents on highway segments (Zegeer et al., 1992; Vogt and Bared, 1998; Poch and Mannering, 1998; Khan et al., 1999; Sayed and Rodriguez, 1999; Harwood et al., 2000). Among them, Vogt and Bared's (1998) and Zegeer et al.'s (1992) models are most popularly used in highway design models; for instance, HERS-ST, ISHDM, and Jha (2000) adopt Vogt and Bared's model for predicting accidents on two-lane rural highways, and Zegeer et al.'s model is used in Jong's (1998) alignment optimization model for estimating accident cost.

However the application of such accident models for evaluating safety improvements in the highway network from adding a new facility is inadequate since they demand various overly detailed inputs. The independent variables that are commonly used in those accident models require detailed site-specific information, which is generally not available at the planning stage (Chatterjee et al., 2003). Obtaining detailed highway geometric variables (such as fraction of total segment lengths occupied by individual horizontal and vertical curves and absolute change in grade in the highway) for all highways in a given highway network is very expensive; furthermore, they may not be significantly meaningful for evaluating safety improvements from a project.

Several accident prediction models have been developed for planning purposes (Chatterjee et al., 2003; Persaud, 1991; Poole and Cribbins, 1983). These models predict accident frequency based on independent variables for which data are generally available in the planning stage, such as annual average daily traffic (AADT), volume to capacity ratio (V/C), and road length. This dissertation employs Chatterjee et al.'s (2003) model to predict number of accidents on a highway segment since the model can consider different highway functional types (e.g., freeway, divided multilane highways, and two-lane highways) besides the traffic volumes operating on that segment and its length. Table 7.10 shows Chatterjee et al.'s accident prediction models for four different highway classes.

Table 7.10 Accident Prediction Models by Chatterjee et al. (2003)

Segment Type	Accident Prediction Function
Freeways	$tot = \exp\begin{pmatrix} 2.547329 + 0.0137348 \times Kaadt \\ +0.654683 \times seg - length \end{pmatrix} - 1$
Undivided Highways	$tot = \exp\begin{pmatrix} 1.843907 + 0.0315530 \times Kaadt \\ +1.56333 \times seg - length \end{pmatrix} - 1$
Divided Highways	$tot = \exp\begin{pmatrix} 2.045214 + 0.0253678 \times Kaadt \\ +0.974015 \times seg - length \end{pmatrix} - 1$
Two Lane Highways	$tot = \exp\begin{pmatrix} 1.742611 + 0.0451726 \times Kaadt \\ +0.700194 \times seg - length \end{pmatrix} - 1$

where *Kaadt* equals, 1000 $\times$daily assigned link volume, *tot* equals total accidents predicted for 3 years, and *seg-length* equals link length (mi)

Let us recall our optimization problem for a small highway network. There are more than two highway segments in the given road network, and this number can increase if a new highway is added to the network, In addition, there may be several types of highways such as freeways, undivided highways, and two-lane highways in the network. Taking all these considerations into account, we now rewrite Chatterjee et al.'s (2003) model as follows; note that the 3-year accidents totals (*tot*) predicted from Chatterjee et al.'s model can be annualized by dividing the values by 3 and we can also account for link directionality after further dividing by 2:

$$\mathbf{Acc}_a \cdot \mathbf{I}_a = \begin{bmatrix} A_a^{Fre} \\ A_a^{Und} \\ A_a^{Div} \\ A_a^{Two} \end{bmatrix} \cdot \begin{bmatrix} I_a^{Fre} \\ I_a^{Und} \\ I_a^{Div} \\ I_a^{Two} \end{bmatrix} \qquad \forall a \in \boldsymbol{A} \tag{7.20}$$
$$= \frac{1}{6} \begin{bmatrix} \exp\left(2.547329 + 0.013735\left(0.001 AADT_a\right) + 0.654683 L_a\right) - 1 \\ \exp\left(1.843907 + 0.031553\left(0.001 AADT_a\right) + 1.563330 L_a\right) - 1 \\ \exp\left(2.045214 + 0.025368\left(0.001 AADT_a\right) + 0.974015 L_a\right) - 1 \\ \exp\left(1.742611 + 0.045173\left(0.001 AADT_a\right) + 0.700194 L_a\right) - 1 \end{bmatrix} \cdot \begin{bmatrix} I_a^{Fre} \\ I_a^{Und} \\ I_a^{Div} \\ I_a^{Two} \end{bmatrix}$$

where: $\mathbf{Acc}_a$ = Vector representation of average annual accidents (accidents/yr) on arc a of which functional type is either freeway, undivided-multilane-highway, divided multilane-highway, or two-lane-highway, respectively; $\mathbf{Acc}_a = \left[A_a^{Fre} \; A_a^{Und} \; A_a^{Div} \; A_a^{Two} \right],\ a \in \boldsymbol{A}$

$\boldsymbol{A}$ = A set of arcs in a given highway network

$\mathbf{I}_a$ = Vector representation of dummy variables indicating functional type of highway arc a; $\mathbf{I}_a = \left[I_a^{Fre} \; I_a^{Und} \; I_a^{Div} \; I_a^{Two} \right],\ a \in \boldsymbol{A}$

$$I_a^{Fre} = \begin{cases} 1 & \text{if arc } a \text{ is a freeway} \\ 0 & \text{otherwise} \end{cases},$$

$$I_a^{Und} = \begin{cases} 1 & \text{if arc } a \text{ is an undivided-multilane highway} \\ 0 & \text{otherwise} \end{cases},$$

$$I_a^{Div} = \begin{cases} 1 & \text{if arc } a \text{ is a divided-multilane highway} \\ 0 & \text{otherwise} \end{cases},$$

$$I_a^{Two} = \begin{cases} 1 & \text{if arc } a \text{ is a two-lane highway} \\ 0 & \text{otherwise} \end{cases}$$

$AADT_a$ = Annual average daily traffic on arc a; see equation (7.21)

L_a = Length of arc a

We now estimate the accident cost on each road segment in the network with the number of accidents predicted with equation (7.20) and with the accident unit cost obtained from Table 7.9, which is \$53,900 per accident. The following steps are used for estimating the total accident cost saving from the new highway construction in the optimization process:

STEP 1: Update road-lengths and traffic volumes of all highways from the traffic assignment process

$$L_{a^0}, L_{a^1}, \text{ and } \begin{cases} \text{for } AM \text{ peak} & : x_{a^0}^{AM} \text{ and } x_{a^1}^{AM} \\ \text{for } PM \text{ peak} & : x_{a^0}^{PM} \text{ and } x_{a^1}^{PM} \\ \text{for } OFF \text{ peak} & : x_{a^0}^{OFF} \text{ and } x_{a^1}^{OFF} \end{cases} \quad \forall a^0 \in \boldsymbol{A}^0 \text{ and } \forall a^1 \in \boldsymbol{A}^1$$

$$AADT_{a^1} = \left(x_{a^1}^{AM} H_{AM} \right) + \left(x_{a^1}^{PM} H_{PM} \right) + \left(x_{a^1}^{OFF} \left(24 - (H_{AM} + H_{PM}) \right) \right) \qquad (7.21a)$$

$$AADT_{a^0} = \left(x_{a^0}^{AM} H_{AM} \right) + \left(x_{a^0}^{PM} H_{PM} \right) + \left(x_{a^0}^{OFF} \left(24 - (H_{AM} + H_{PM}) \right) \right) \qquad (7.21b)$$

where: $\boldsymbol{A}^0$ = A set of arcs in the existing road network

$\boldsymbol{A}^1$ = A set of arcs in the updated network after a new highway addition

$H_{AM} = AM$ peak duration (hrs) per day

$H_{PM} = PM$ peak duration (hrs) per day

L_{a^0} and L_{a^1} are lengths of arc a^0 and a^1, respectively.

$AADT_{a^1}$, $AADT_{a^0}$ are annual average daily traffic on arc a^1 and a^0, respectively.

Note that $x_{a^0}^{AM}$, $x_{a^0}^{PM}$, and $x_{a^0}^{OFF}$ are computed only once at the beginning of the optimization process.

STEP 2: Calculate total accident cost over the highway network

$$C_{A\_B}^1 = U_{Acc} \sum_{a^1 \in \boldsymbol{A}^1} \mathbf{Acc}_{a^1} \cdot \mathbf{I}_{a^1} \tag{7.22a}$$

$$C_{A\_B}^0 = U_{Acc} \sum_{a^0 \in \boldsymbol{A}^0} \mathbf{Acc}_{a^0} \cdot \mathbf{I}_{a^0} \tag{7.22b}$$

where: $C_{A\_B}^1$ = Total accident cost (\$/year) for traffic operating on all highways over the network with a new highway construction in the base year

$C_{A\_B}^0$ = Total accident cost (\$/year) for traffic operating on all highways over the network with no highway construction in the base year

U_{Acc} = Accidet unit cost (\$/accident) from net perceived value in Table 7.9

$\mathbf{Acc}_{a^1}$, $\mathbf{Acc}_{a^0}$ = Average accidents (accidents/year) on arc a^1 and a^0, respectively; $a^1 \in \boldsymbol{A}^1$, $a^0 \in \boldsymbol{A}^0$; also refer to equation (7.20)

$\mathbf{I}_{a^1}$, $\mathbf{I}_{a^0}$ = Vector representation of dummy variables indicating functional type of arc a^1 and a^0, respectively

STEP 3: Calculate present value of total accident cost saving

$$\Delta C_A = \left(C_{A\_B}^0 - C_{A\_B}^1\right) \times \left[\frac{e^{(r_t - \rho)n_y} - 1}{r_t - \rho}\right] \tag{7.23}$$

where: ΔC_A = Present value of total accident cost saving after a new highway-construction for the analysis period n_y and other parameters are as defined earlier.

Note that the total vehicle mile traveled over the network may generally increase after the new highway construction since the construction scenario entails more highway segments in that network. Thus, it seems that the new highway construction project leads to more traffic accidents over the network. However, the average accident cost per vehicle-mile would be expected to decrease from the new highway addition since the per-unit cost of travel can decrease as the network improves (e.g., as the V/C ratio of highway segments decreases).

7.2.3.2 Accident Cost for At-grade Intersections

Besides basic highway segments, intersections may also cause traffic accidents. Intersection entering volume (to major and minor roads), intersection control types (e.g., pre-timed, stop controlled, and actuated), and density of surrounding driveways are typically considered for major factors accounting for the intersection accidents. Thus, if there are intersections in the given highway network, the intersection accident cost should also be included in the total accident cost estimation procedure presented above.

Substantial research has been devoted to developing models for estimating accident costs on at-grade intersections (Chatterjee et al., 2003; Sayed and Rodriguez, 1999; Khan et al., 1999; Vogt and Bared, 1998; Poch and Mannering, 1998; Lau and May; 1988). Among them, this section only introduces two useful intersection accident-prediction models that could be properly used at a highway planning stage. Lau and May's models (1988) are employed for predicting accidents in signalized intersections, while Vogt and Bared's (1998)[18] models are used for predicting those in un-signalized stop-controlled intersections. Other models may be substituted for these selected accident prediction models if they are considered to be good enough to use in the optimization model. Lau and May's and Vogt and Bared's intersection accident models are described as follows:

Accident prediction on signalized intersections (based on Lau and May,1988):

$\forall i \in \boldsymbol{N}_{IS}$, if node i is a *"4-leg signalized intersection"*, then

$$A_i^{sig4} = \begin{cases} 0.61856 + 0.16911\left(AADT_{i\_1} + AADT_{i\_2}\right)\left(365/10^6\right) + 0.77; \text{ if } n_c \geq 4 & (7.24a) \\ 0.61856 + 0.16911\left(AADT_{i\_1} + AADT_{i\_2}\right)\left(365/10^6\right) + 0.05; \text{ otherwise} & (7.24b) \end{cases}$$

$\forall i \in \boldsymbol{N}_{IS}$, if node i is a *"3-leg signalized intersection"*, then

$$A_i^{sig3} = 0.61856 + 0.16911\left(AADT_{i\_1} + AADT_{i\_2}\right)\left(365/10^6\right) - 0.62 \qquad (7.25)$$

where: $\boldsymbol{N}_{IS}$ = A set of (at-grade) intersection nodes in a given highway network

A_i^{sig4} = Predicted intersection accidents (accidents/yr) if node i is a "4-Leg signalized intersection"

A_i^{sig3} = Predicted intersection accidents (accidents/yr) if node i is a "3-Leg signalized intersection"

$AADT_{i\_1}$ = Annual average daily traffic entering to node i from crossroad 1

$AADT_{i\_2}$ = Annual average daily traffic entering to node i from crossroad 2

n_c = Number of lanes on crossroad at node i

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[18] Vogt and Bared's (1998) intersection accident model is used as a base model to predict intersection accidents in the Interactive Highway Safety Design Model (IHSDM) by FHWA; IHSDM is an analysis tool for evaluating safety and operational effects of geometric design decisions on two-lane rural highways.

Accident prediction on unsignalized intersections (based on Vogt and Bared, 1998):

$\forall i \in \boldsymbol{N}_{IS}$, if node i is a *"4-leg stop-controlled intersection"*, then

$$A_i^{unsig4} = \exp\left[-9.34 + 0.60\ln\left(AADT_{i\_1}\right) + 0.61\ln\left(AADT_{i\_2}\right)\right] \tag{7.26}$$

$\forall i \in \boldsymbol{N}_{IS}$ if node i is a *"3-leg stop-controlled intersection"*, then

$$A_i^{unsig3} = \exp\left[-10.9 + 0.79\ln\left(AADT_{i\_1}\right) + 0.49\ln\left(AADT_{i\_2}\right)\right] \tag{7.27}$$

where: A_i^{unsig4} = Predicted intersection accidents (accidents/yr) if node i is a "4-Leg unsignalized intersection"

A_i^{unsig3} = Predicted intersection accidents (accidents/yr) if node i is a "3-Leg unsignalized intersection"

Let $\mathbf{Acc}_i$ be the vector of predicted annual accidents (accidents/year) for at-grade intersection i, and $\mathbf{I}_i$ be the vector of dummy variables indicating type of intersection i (whether signalized or un-signalized and whether 4-leg or 3-leg intersections). Then, we may rewrite the above models with the following simple formula:

$$\mathbf{Acc}_i \cdot \mathbf{I}_i = \begin{bmatrix} A_i^{sig4} \\ A_i^{sig3} \\ A_i^{unsig4} \\ A_i^{unsig3} \end{bmatrix} \cdot \begin{bmatrix} I_i^{sig4} \\ I_i^{sig3} \\ I_i^{unsig4} \\ I_i^{unsig3} \end{bmatrix} \quad \forall i \in \boldsymbol{N}_{IS} \tag{7.28}$$

where: $\mathbf{Acc}_i$ = Average intersection accidents (accidents/yr) at intersection i of which type is either 4-leg signalized, 3-leg signalized, 4-leg stop controlled, or 3-leg stop controlled intersection, respectively; $\mathbf{Acc}_i = \left[A_i^{sig4}\ A_i^{sig3}\ A_i^{unsig4}\ A_i^{unsig3}\right]$, $i \in \boldsymbol{N}_{IS}$

$\mathbf{I}_i$ = Vector representation of dummy variables indicating type of at-grade intersection i; $\mathbf{I}_i = \left[I_i^{sig4}\ I_i^{sig3}\ I_i^{unsig4}\ I_i^{unsig3}\right]$, $i \in \boldsymbol{N}_{IS}$

$$I_i^{sig4} = \begin{cases} 1 & \text{if node } i \text{ is "4-Leg signalized intersection"} \\ 0 & \text{otherwise} \end{cases},$$

$$I_i^{sig3} = \begin{cases} 1 & \text{if node } i \text{ is "3-Leg signalized intersection"} \\ 0 & \text{otherwise} \end{cases},$$

$$I_i^{unsig4} = \begin{cases} 1 & \text{if node } i \text{ is "4-Leg un-signalized intersection"} \\ 0 & \text{otherwise} \end{cases},$$

$$I_i^{unsig3} = \begin{cases} 1 & \text{if node } i \text{ is "3-Leg un-signalized intersection"} \\ 0 & \text{otherwise} \end{cases}$$

We now can estimate the accident cost at each intersection in the highway network with the number of intersection accidents (**Acc**$_i$) predicted with equation (7.28) and with the accident unit cost (U_{ACC}) from Table 7.9. Note that the procedure for calculating the total intersection-accident cost saving, which also can be obtained from the network improvement, is almost the same as that for the accident cost saving on the basic highway segments (in Section 7.2.3.1).

PART III: CASE STUDIY AND SUMMARY

The theoretical background of the proposed model is described in the preceding parts; Part I presents efficient solution search methods required for the model to be an effective alignment optimization model, and Part II further extends the model capabilities to a simple highway network optimization, by reformulating the model structure as a bi-level programming problem.

Part III discusses applicability and usability of the model. In Chapter 8 model application to two real highway projects are described and key findings are discussed. Finally, research contributions and recommendations for future extensions of the model are summarized in Chapter 9.

Chapter 8: Case Study and Discussion

This chapter consists of two example studies of the proposed alignment optimization model for demonstrating its capability and usability in model applications to real highway construction projects. The sensitivity of the optimized alignments found by the model with various input parameters as well as their goodness tests are comprehensively investigated.

In the first example, the alignment optimization model is employed to search for an optimal bypass of a congested local highway. The best alignments of the new bypass connecting two pre-specified endpoints (located on the upstream and downstream of the congested section of the road) are searched given the detailed geographical and design inputs associated with highway construction (such as design speed, road width, and ground elevation of the study area). In the second example, the model alignment optimization capabilities are tested for a simple highway network. The locations of the start and end points of the new highway are not given in this example. Instead, their optimal locations are optimized simultaneously with the rest of the alignment. In addition, for finding the best highway alternatives, system improvements due to the new highway addition to an existing road network (such as network users' travel time and vehicle operation costs savings) are also considered in the model objective function besides the various highway agency cost components considered in the Brookeville case study (e.g., earthwork, right-of-way, and road pavement costs). As stated earlier, the model structure is reformulated as a bi-level programming problem for this application (see Chapter 6).

Note that various geographical constraints (i.e., spatially untouchable and/or partially untouchable areas) as well as design constraints (e.g., minimum sight distance and maximum gradient) associated with the road construction project are also considered in both the case studies. In order to asses how good the solutions found from the model are, we compare solution alignments found from a random search process and those from the proposed model. The model capability is also demonstrated with outputs from the sensitivity analysis to various critical model parameters (such as the number of PI's, composition of objective function, design speed, and analysis period).

8.1 Case Study 1 (Maryland Brookeville Bypass Example)

This case study is organized as follows: After the problem description in section 8.1.1, model application procedures including required inputs and detailed data preprocessing tasks are described in the next section (8.1.2). GIS map digitization and trade-offs in map representation required for automated right-of-way cost and environmental impact estimation are also covered in this section. In the third section (8.1.3), optimized alignments found with different input PI's are presented along with a statistical test for assessing the goodness of the solutions. The application results showing the sensitivity of the optimized alignments to various model parameters are presented in the final section (8.1.4).

8.1.1 Problem Description

The Maryland State Highway Administration (MDSHA) has been working on the MD 97 Brookeville bypass project in Montgomery County, Maryland. This area is listed on the National Register of Historic Places as a historic district, and is located in approximately ten miles south of I-70 and three miles north of MD 108. The project objectives are to divert the increasing traffic volumes from the town of Brookeville by constructing a new bypass route so as to improve traffic operation and safety on existing MD 97, while preserving the historic character of the town. The alignment optimization model is tested in the real highway construction project to assist the local government for finding the best alternatives while considering various issues arising in the project.

Through this case study we (i) demonstrate the applicability and usability of the model to a real highway project with due consideration to issues arising in real-world applications and (ii) analyze the sensitivity of solution alignments to various user-specified input variables (such as the number of points of intersection (PI's), composition of the model objective function, and design speed); in addition, (iii) goodness of the solutions found from the model is statistically evaluated.

To ensure comparability with the normal evaluation criteria typically used by the highway agencies, such as those used by the MDSHA, the user cost which consists of travel time cost, vehicle operating cost, and the accident cost is suppressed from the model objective function. Thus, the objective function used in applying the model to the Brookeville project is $C_{T\_Agency} = C_L + C_R + C_E + C_S + C_M$.

8.1.2 Data and Application Procedure

Three major data preprocessing tasks are performed before optimizing highway alignments with the model; (i) horizontal map digitization, (ii) vertical map digitization, and (iii) tradeoff in map representation. Figure 8.1 presents the application procedure of the model to the Brookeville Bypass project. Maryland's GIS database (MDProperty View) and the Microstation base maps for Brookeville area (from MDSHA) are used to construct the study area.

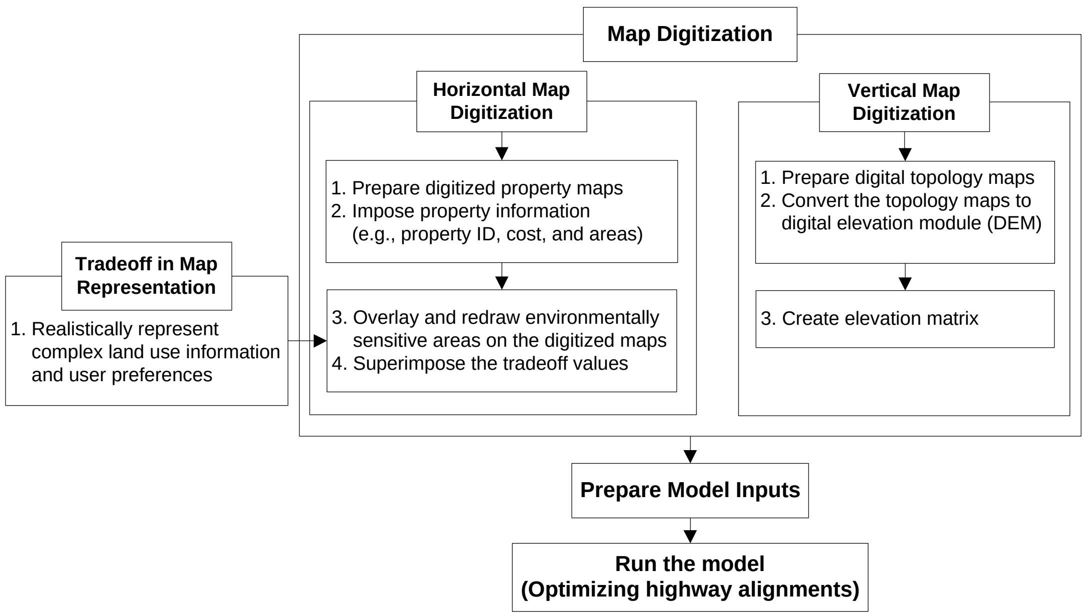


Figure 8.1 Model Application Procedures for the Brookeville Example

Horizontal Map Digitization

For horizontal map digitization, Microstation base-maps which store boundaries of environmentally sensitive areas, such as wetlands, floodplains, and historic resources are used to digitize properties in the study area of Brookeville. In this step, each property is regarded as a polygon, which can retain property information as its attributes. The purpose of horizontal map digitization is to reflect complex land-uses in the study area on the GIS digitized map, and eventually to use it for evaluating the detailed alignment right-of-way cost and environmental impacts during the optimization process. The information assigned on the map includes parcel ID number, perimeter, unit cost, and area of each property.

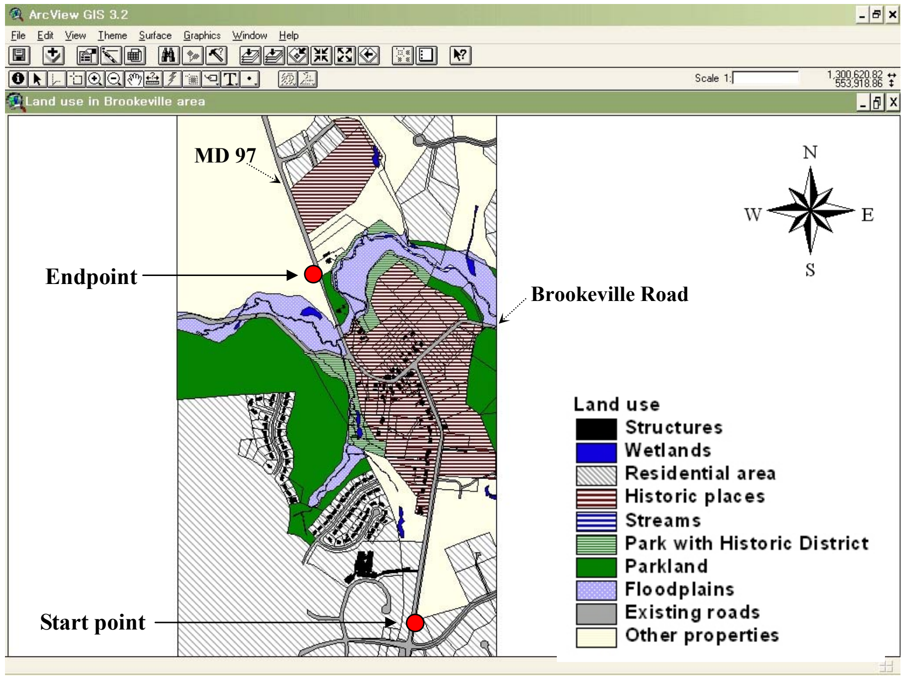
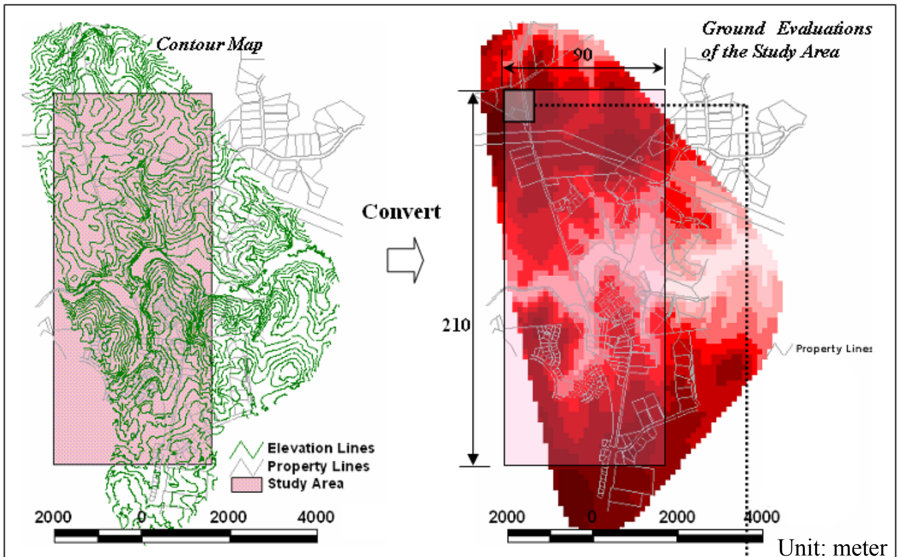


Figure 8.2 Land Use of the Study Area for Brookeville Example

As shown in Figure 8.2, the study area combines various types of natural and cultural land-use patterns. There are 10 different types of land-use characteristics in the study area: structures (houses and other facilities), wetlands, residential areas, historic places, streams, park with historic district, parklands, floodplains, existing roads, and other properties. Note that such a map superimposition is pre-processed with the IDPM, and is essential for applying the feasible gates (FG) methods which is designed for representing the user preferences in the alignment optimization process effectively. Through preprocessing, the model users can define feasible bounds of solution alignments generated by the model (see Chapter 3 for the details of the FG methods). The study area comprises about 650 geographic entities (including land, structures, road etc.) with given start and end points of the proposed alignment. The 690 acres (2.792 km^2) of the search space includes 203.3 acres (0.823 km^2) of primarily residential areas, 73.4 acres (0.297 km^2) of historic sites, 67.5 acres (0.273 km^2) of parkland, and 30.9 acres (0.125 km^2) of floodplains.



Y grids \ X grids	1	2	3	4	5	..	90
1	143	143	143	141	141	..	132
2	143	143	143	142	141	..	132
3	143	143	143	142	140	..	133
4	143	143	143	141	140		133
5	144	143	142	141	140	...	134
. .	. .	. .	. .	. .	. .	... ...	. .
210	123	121	119	120	122	...	141

Figure 8.3 Ground Elevation of the Brookeville Study Area

Vertical Map Digitization

In the model, the alignment earthwork cost is calculated based on a ground elevation, whose preparation for the study area is required. To do this, we use a Microstation contour map for the study area, and convert it to a Digital Elevation Model (DEM) that provides elevations with a grid base as shown in Figure 8.3. The study area is divided into evenly spaced grids of 40feet×40feet (12meters×12meters). Finer grids may be selected for precise earthwork calculation if desired. The elevation range in the Brookeville area is 328 to 508 feet (100 to 155 meters). The darker areas represent higher elevations. Floodplains and parklands exist in low elevation areas while the historic places are located at relatively high elevations (refer to Figures 8.2 and 8.3).

Tradeoffs in Map Representation for Environmental Issues

When considering roadway construction in a given project area, various geographically sensitive regions (such as historic sites, creeks, public facilities, etc.) may be encountered. These control areas should be avoided by the proposed alignment, whose impact on these regions should be minimized as much as possible. Based on a previous Brookeville study by MDSHA (2001), we recognize residential properties, the Longwood Community center, historic districts, and wetlands as environmentally primary sensitive areas that should be avoided by the new alignments if at all possible (i.e., those are untouchable areas). In addition, parklands, floodplains, and streams, which are located between the given start and end points so as to unavoidably be taken by the proposed alignment, are considered environmentally secondary sensitive areas.

To realistically represent such control areas in the model application, we divide them into two categories based on their land-use characteristics as shown in Table 8.1: Type1 areas that the proposed roadway alternatives can avoid, and Type2 areas that the proposed alternatives cannot avoid. Type1 areas include wetlands, historic places, residential areas, Community Center, and other structures. Type2 areas consist of streams, parklands and floodplains, which are unavoidably affected by the alignments.

To properly reflect these relevant environmental issues in the GIS map representation, tradeoff values with respect to the different land use types must be carefully determined based on their relative importance, since these values can significantly affect the resulting alignment. Thus, the maximum allowable areas affected by the new alignments (denoted as *MaxA*) should be much stricter for Type1 areas than for Type2 areas; recall that Type1 areas have primary (i.e., stronger) environmental regions to be avoided by the alignments whereas Type2 areas contain only secondary regions. Note that this idea seeks to eliminate the alignments' impacts on Type1 areas and minimize those on Type2 areas, by guiding the alignments to take other properties, which have no restrictions. For this purpose, we discriminate between Type1 and Type2 areas by assigning different values of *MaxA*. For the environmentally sensitive regions classified as Type1 areas, their *MaxA* are set to be 0 (which means Type1 areas are not allowed to be affected by the new alignments), while the *MaxA* of control areas defined as Type2 can be interactively specified by the model users based on their relative importance.

Table 8.1 Spatial Control Areas in the Brookeville Example

Type	Control areas	Characteristics	*MaxA*
Type1	Wetlands, historic places, residential properties, site of community center, structures (houses, public facilities, etc.)	The control areas that the proposed alignment can avoid	0
Type2	Streams, floodplains, parklands	The control areas that the proposed alignment cannot avoid	User-specifiable

Description of Model Inputs and Outputs

For optimizing highway alignments with the model, some input variables must be pre-specified. These are, for instance, road width, design speed, and maximum vertical gradient of the proposed alignments. Since the optimized alignment varies depending on these inputs, users should carefully determine the input variable values. The start and end points of the new alignments are assumed to be known in this case study. They are located on the south and north sections of MD 97 in Brookeville, respectively, as shown in Figure 8.2. The Euclidean distance between the start and end points is about 0.76 mile (1.22 km). The design speed was initially set at 50 mph (80kph). The distances between station points (i.e., cross-section spacing), which are used as earthwork computation unit in the model formulation, are assumed to be 50 feet (15 meters).

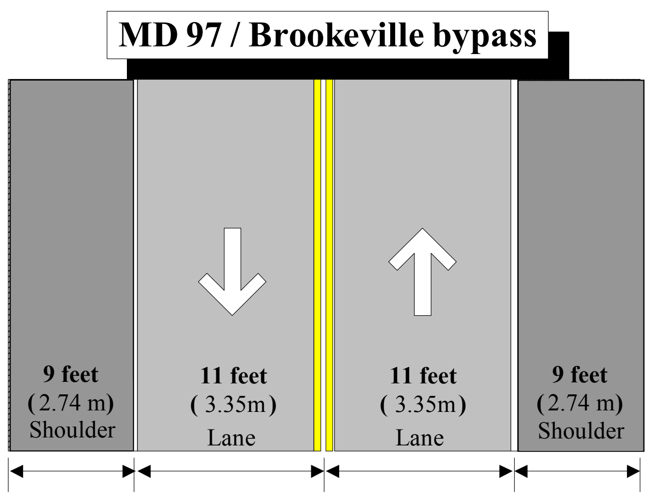


Figure 8.4 Cross Section of the New Alignment for Brookeville Example

The cross-section of the proposed alignment is assumed to represent a 2-lane road with a 40 foot width (11 feet for lanes and 9 feet for shoulders as shown in Figure 8.4). In addition, grade separation is assumed to be the only crossing type of the new highway with the existing Brookeville Road. Various user-specifiable input variables required in the highway alignment optimization process are described on the left side of Table 8.2. The values on the right side of the table are used for the model application to the Brookeville example. The unit road construction costs, such as unit cut and fill costs and length- dependent costs are user-specifiable. The total cost of a solution alignment is computed based on the pre-specified unit costs.

Table 8.2 Baseline Inputs Used in the Model Application to Brookeville Example

Input variables	Value
No. of intersection points (PI's)	4 ~ 7
Road width	40 foot, 2-lane road (11"lane, 9"shoulder)
Design speed	50 mph (80 kph)
Maximum superelevation	0.06
Maximum allowable grade	5 %
Coefficient of side friction	0.16
Longitudinal friction coefficient	0.28
Distance between station points	50 feet (15 meters)
Fill slope	0.4
Cut slope	0.5
Earth shrinkage factor	0.9
Unit cut cost	35 $\$/yard^3$ (45.5 $\$/m^3$)
Unit fill cost	20 $\$/yard^3$ (26 $\$/m^3$)
Cost of moving earth from a borrow pit	2 $\$/yard^3$ (2.6 $\$/m^3$)
Cost of moving earth to a fill	3 $\$/yard^3$ (3.9 $\$/m^3$)
Unit length-dependent cost[19]	400 $/feet (656 $/meter)
Crossing type with the existing road	Grade separation
Terrain height ranges	328 ~ 508 feet (100 ~ 155 meter)
Unit land value in the study area	0 ~ 14 $\$/ft^2$ (0 ~ 151 $\$/m^2$)

Note that detailed results for the optimized alignments, such as total cost breakdown, earthwork cost per station, and coordinates of all evaluated alignments are provided as the model outputs. These results are automatically restored in different files during program runs. In addition, alignments' impacts to the environmentally sensitive areas can also be summarized using the GIS module embedded in the optimization model.

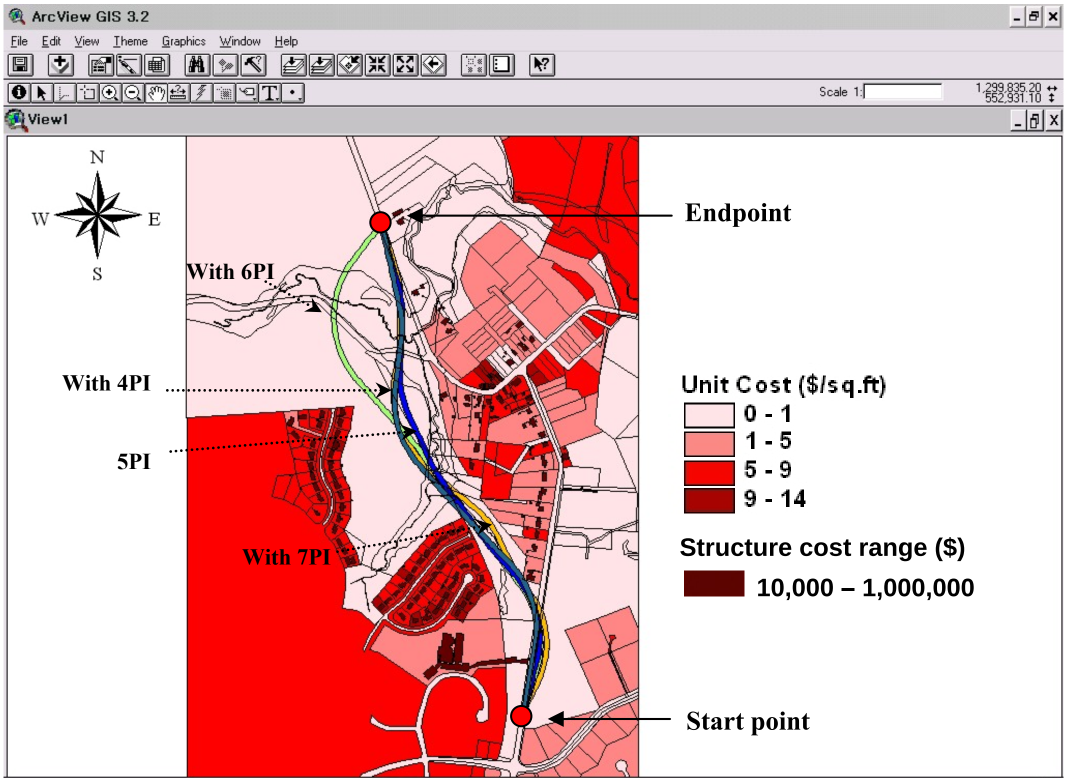
[19] Unit length-dependent cost mainly consists of unit pavement cost and sub and super structure (e.g. barrier and median) costs on the road

8.1.3 Optimization Results

8.1.3.1 Optimized Alignments with Different Number of PI's

Optimizing (roughly) the number of PI's is quite desirable in a model application to real highway construction projects because more PI's may be preferable for alignments in complex and high density areas (such as urban areas), while fewer PI's may suffice for projects in the rural study areas. For instance, applying the lower number of PI's (e.g., 2 or 3 PI's) to the Brookeville example may not be sufficient to keep solution alignments away from the complex control areas. It should be also noted that the solution quality (such as alignment's impact on the environmentally sensitive areas and right-of-way cost) and computation efficiency of the model may vary depending on the number of PI's.

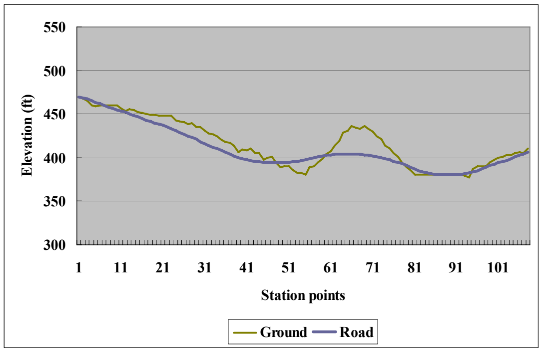
The optimized solution alignments found with the inputs presented in Table 8.2 are shown in Figures 8.5 and 8.6. To explore the preferable number of PI's, the model was run four times with 4 to 7 PI's. As shown in Figure 8.5, horizontal profiles of the optimized alignments A, B, C, D have 4, 5, 6, and 7 PI's, respectively. Vertical profiles of those alignments are presented in Figure 8.6. Note that more than 8 PI's were not considered in this case study since they might create too many horizontal curves and increase the model computation time. For each of the four cases, the model searched over 300 generations, thereby evaluating 6,500 alignments. A desktop PC Pentium IV 3.2 GHZ with 2 GB RAM was used to run the model. It took a considerable time (about 4.5 to 6.5 hours) to run through 300 generations because the Brookeville study area is quite complex and has many properties (about 650 geographical entities).



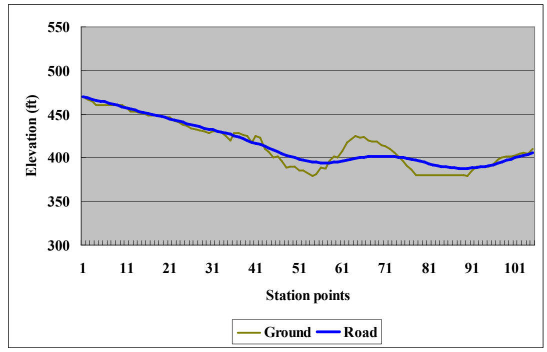
Optimized alignment	# of PI's	Total agency costs ($)	Environmental impact				Length (ft)	Computation time (hr)
			The control area taken by alignments (ft2)			Residential relocation (No.)		
			Type 1	Type 2	Sum			
A	4	4,847,128	458.3	70,674.2	71,132.6	0	4,359.9	4.41
B	5	4,328,432	0.0	63,030.4	63,030.4	0	4,302.0	4.68
C	6	5,655,707	0.0	82,017.4	82,017.4	0	4,607.3	4.95
D	7	4,919,403	0.0	64,489.3	64,489.3	0	4,422.9	5.01

Figure 8.5 Horizontal Profiles of Optimized Alignments Having Different PI's for Brookeville Example

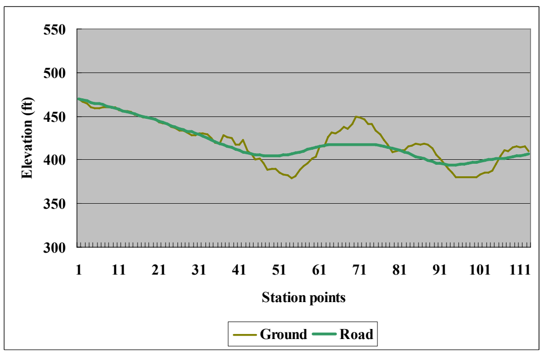
As shown in Figures 8.5 and 8.6, the horizontal and vertical profiles of the four optimized alignments seem to be very similar. They have similar rights-of-way and alignment lengths; in addition, none of the four alternatives require any residential relocation. However, it should be noted that detailed model outputs, such as total agency costs and environmental impacts of those alignments are quite different. Among the four alternatives, the lowest agency cost is found to be $ 4,328,432 and the highest cost is found to be $ 5,655,707.



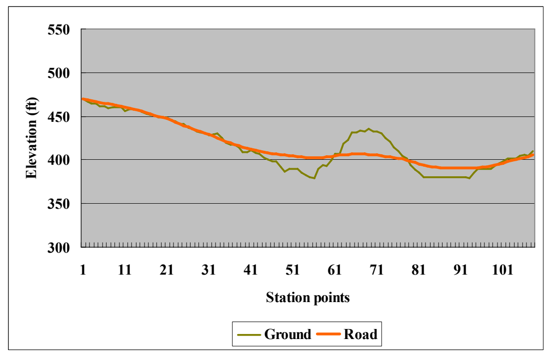
(a) Optimized Vertical Alignment A Having 4 PI's



(b) Optimized Vertical Alignment B Having 5 PI's



(c) Optimized Vertical Alignment C Having 8 PI's



(d) Optimized Vertical Alignment D Having 7 PI's

Figure 8.6 Vertical Profiles of Optimized Alignments Having Different PI's for Brookeville Example

In terms of environmental impact, the sensitive areas taken by the optimized alignment B (63,030.4 ft^2 for total) are the lowest, although the differences among the four alternatives are not great (see Figure 8.5). For Type1 areas, which were previously defined as primary sensitive regions, optimized alignment A with 4 PI's affects relatively large amounts of Type1 areas compared to those of the other three alternatives. Alignment A affects 458.3 ft^2 of Type1 areas (306 ft^2 for residential area and 152.3 ft^2 for Longwood Community Center); the other three optimized alignments do not affect Type1 areas. A more detailed environmental impact summary for the four alternatives is presented in Table 8.3.

Table 8.3 Environmental Impact Summary for Optimized Alignments A to B

Optimized alignments		A	B	C	D
Number of PI's		4	5	6	7
Total agency cost (million $)		4.86	4.33	5.66	4.92
Alignment length (feet)		4,359.9	4,302.0	4,607.3	4,422.9
Socio-economic resources	Residential area affected (ft^2)	305.96	0	0	0
	Residential relocations (no.)	0	0	0	0
	Community center affected (ft^2)	152.38	0	0	0
	Historic places affected (ft^2)	0	0	0	0
	County reserved areas affected (ft^2)	41,896.1	45,295.9	45,286.0	45,260.0
	Existing roads affected (ft^2)	39,152.1	29,609.1	17,037.6	25,227.4
Natural resources	Wetlands affected (ft^2)	0	0	0	0
	Floodplains affected (ft^2)	23,259.8	17,260.3	16,689.7	14,883.5
	Streams affected (ft^2)	690.5	777.6	634.9	610.7
	Parkland affected (ft^2)	46,723.8	44,992.5	64,692.7	48,995.2

In terms of computation efficiency (refer to Figure 8.5), the model computation time increases slightly when the number of PI's increases from 4 to 7. It seems that computation time is not greatly affected by the number of PI's. However, it should be noted that computation time still increases with the number of PI's since additional PI's generate additional horizontal and vertical curved sections. For instance, a model application with 20 input PI's for the same example project requires over 10 hours of computations.

It should be noted that the total agency cost estimated from the model (see Table 8.3) is underestimated. This cost mainly consists of length-dependent, right-of-way, earthwork cost, structure cost, and maintenance cost; i.e., other agency costs required in the road construction (such as drainage landscape architecture cost, traffic signal strain poles cost, etc.) and contingency cost are not included. Readers may refer to Table 3.4 in section 3.4 to see the detailed breakdown of the total agency cost of the optimized alignment found with 5 PI's (i.e., alternative B). In addition, Figure 4.6 presented in section 4.3 shows the changes in objective function values over successive generations for that case.

8.1.3.2 Goodness Test

Although the solution alignments found with the proposed model seems to be reasonable, we aim to evaluate how good the solutions are. For this purpose, an experiment is designed to statistically test the goodness of the solutions found by the model. Table 8.4 describes three different scenarios of the experiment.

In this experiment, the 1st scenario is initiated by randomly generating solutions to the problem (i.e., sample solutions are generated from a random search process). The 2nd scenario is a random search process with human judgments. This scenario is designed for representing a path selection process of a new highway conducted in an actual road construction project. For this purpose, it is assumed that spatial information about no-go areas (i.e., untouchable areas) that new alignments must avoid is already known and that all generated solutions should meet given design constraints. Such a scenario is implemented by applying (i) the feasible gate (FG) method (for identifying the user-defined alignment feasible boundaries) and (ii) prescreening and repairing (P&R) method (for maintaining the required design specifications) to a random search process. In the last (3rd) scenario the proposed optimization model is to used to search for alignments (i.e., search with the customized GAs integrated with the FG and P&R methods). Note that the first two scenarios have no learning procedure during the search process; however, the 3rd scenario is an adaptive search based on the principles of natural evolution and survival of the fittest.

Table 8.4 Three Test Scenarios for Assessing Goodness of Solutions Found from the Model

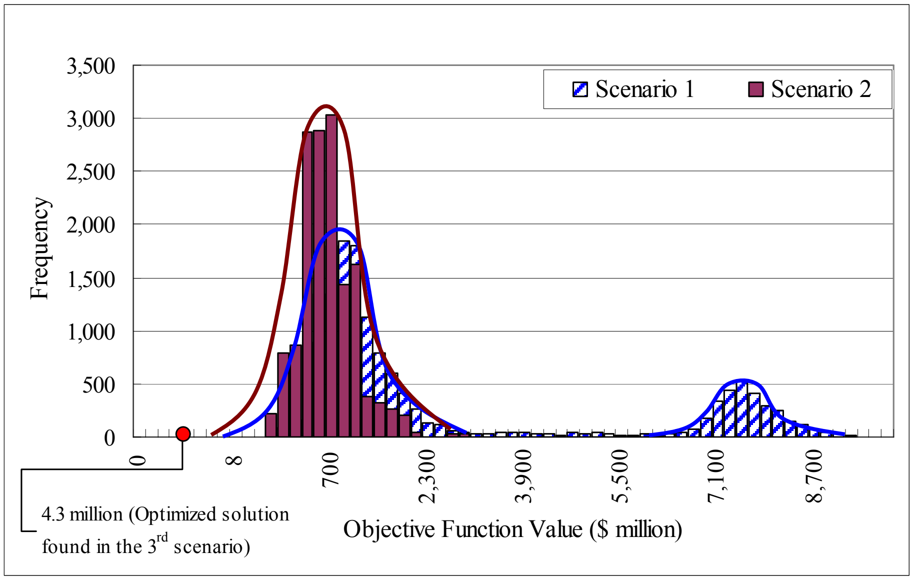
Scenario	Description	Search Option	Iteration
1	Random generation of PI's in the entire study area	Random search	15,000
2	Random generation of PI's within the user-defined feasible bounds	Random search with FG, and P&R methods	15,000
3	Evolutionary search of PI's within the user-defined feasible bounds	GAs-based search with FG and P&R methods	9,050*

* about 9,050 alignments are generated for 300 generations

15,000 of sample solutions are created from each of the 1st and 2nd scenarios. Note that these samples are created in such a way that the solutions are representative and independent of each other. Figure 8.7 shows distribution diagrams of the sample solutions generated from the 1st and 2nd scenarios. In order to ascertain the goodness of the optimized solution found from the 3rd scenario (i.e., optimized alignment B), its relative position is also indicated on that figure.

As shown in Figure 8.7, It is observed that the objective function values of the solution alignments found from the two random search processes (the 1st and 2nd scenarios) are very widely distributed (1st scenario: 18.2 ~ 9,945.0 millions; 2nd scenario: 11.8 ~ 2,802.5 millions) and there are two distinct high frequency ranges in the solution distribution of the 1st scenario case and one in the 2nd scenario case.

(a) Distribution of Objective Function Values



(b) Descriptive Statistics of Objective Function Values

Scenario	Min	Max	Mean	Median	Standard Deviation
1	18.2	9,945.0	2,355.7	1041.7	2,699.4
2	11.8	2,802.5	547.9	493.8	412.8
3	4.3*				

* objective function value (i.e., total agency cost) of the optimized alignment B

Figure 8.7 Experiment Testing the Goodness of the Optimized Solution

Our interpretation for the 1st scenario result is that very high cost properties (such as, house and building structures) are spatially distributed (scattered) only in specific regions of the study area as shown in Figure 8.5, and thus some fractions of alignments generated from the random search processes (i.e., the 1st scenario) can possibly cross those properties but some may not. That is why there are two distinct high frequency ranges in the distribution of the 1st scenario. On the other hand, rights-of-way of alignments resulting from the 2nd scenario are mainly placed on the other properties (e.g., farm and parklands) whose costs are relatively low and range widely, while avoiding the high cost structures. This occurs because the 2nd scenario is designed to perform a random search process within a feasible boundary that represents user preferences (see Figure 3.8 in section 3.4 for the user-specified feasible boundary). However, it should be noted that their objective function values are still higher than those from the optimization model since other cost components included in the objective function (such as, earthwork and length-dependent costs) increase the total objective function values without being optimized.

Figure 8.7 also shows that the objective function value (about 4.3 millions) of the optimized alignment from the model is considerably lower than the lower bounds (18.2 and 11.8 millions) of the sample distributions from the two random search scenarios. This indicates that it dominates all possible solutions in the sample distribution. Thus, the solution found by the model is remarkably good when compared with other possible solutions to the problem.

8.1.4 Alignment Sensitivity to Other Major Input Parameters

Beyond the number of PI's, the sensitivity to other major input parameters of the alignment optimization model (such as components of objective function, design speed, elevation grid size, and cross-section spacing) is also examined in this section. To check the influence of such factors on the solution quality, the input data values implemented for optimized alignment B (see inputs in Table 8.2 with 5 PI's) are used as the default values since it seems the preferable one according to the results presented in Table 8.3; its initial construction cost is the lowest and it does not affect any spatially sensitive area.

8.1.4.1 Sensitivity to Model Objective Function

The sensitivity of optimized alignments to various cost components associated with alignment construction is tested here. This analysis is intended to show the effect of various model objectives so as to emphasize that all the alignment-sensitive costs should be considered and precisely formulated for a good highway optimization model. Three different scenarios are designed to show how each cost component affects the resulting alignments. Note that all the solution alignments found in this sensitivity analysis adopt the same input parameters besides the cost components composing of the objective function as follows:

- Case 1: $C_T=C_L+C_S+C_M$ (i.e., consider length-dependent, structure, and maintenance costs for the objective function)
- Case 2: $C_T=C_L+C_S+C_M+C_R$ (i.e., add right-of-way cost to Case 1)
- Case 3: $C_T= C_L+C_S+C_M+C_R+C_E$ (i.e., add earthwork cost to Case 2)

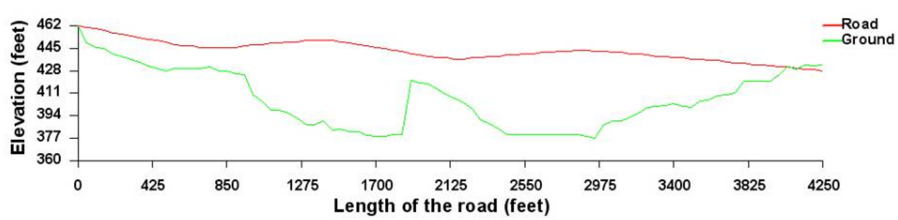
As shown in Figure 8.8(a1), the solution alignment being optimized with only $C_L+C_S+C_M$ is a straight line horizontally and affects many high-cost and environmentally sensitive areas (e.g., residential and historic areas). In addition, Figure 8.8(b1) shows that its vertical profile (i.e., road elevation) is hugely different from the corresponding ground elevation profile and obviously not optimized. Such results occur because the solution alignment is optimized with the objective function that does not represent the complexity of land use system and topography of the study area. Note that the objective function of this case does not include the right-of-way cost, environmental impacts on the sensitive areas, and earthwork cost.

(a1) Optimized Alignment with $C_T = C_L + C_S + C_M$ (Case1)

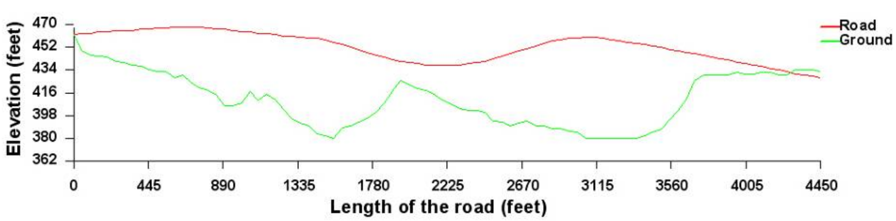
(a2) Optimized Alignment with $C_T = C_L + C_S + C_M + C_R$ (Case2)

(a3) Optimized Alignment with $C_T = C_L + C_S + C_M + C_R + C_E$ (Case3)

(b1) Vertical Profile of Optimized Alignment for Case1



(b2) Vertical Profile of Optimized Alignment for Case2



(b3) Vertical Profile of Optimized Alignment for Case3

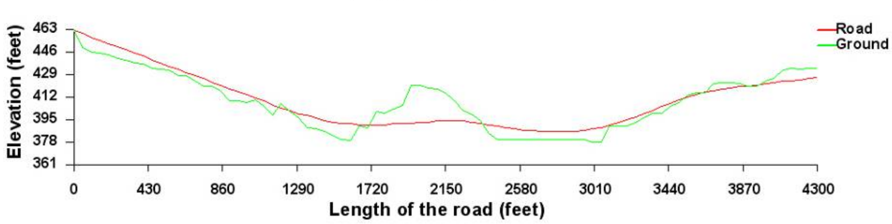


Figure 8.8 Sensitivity of Optimized Alignments to Objective Function

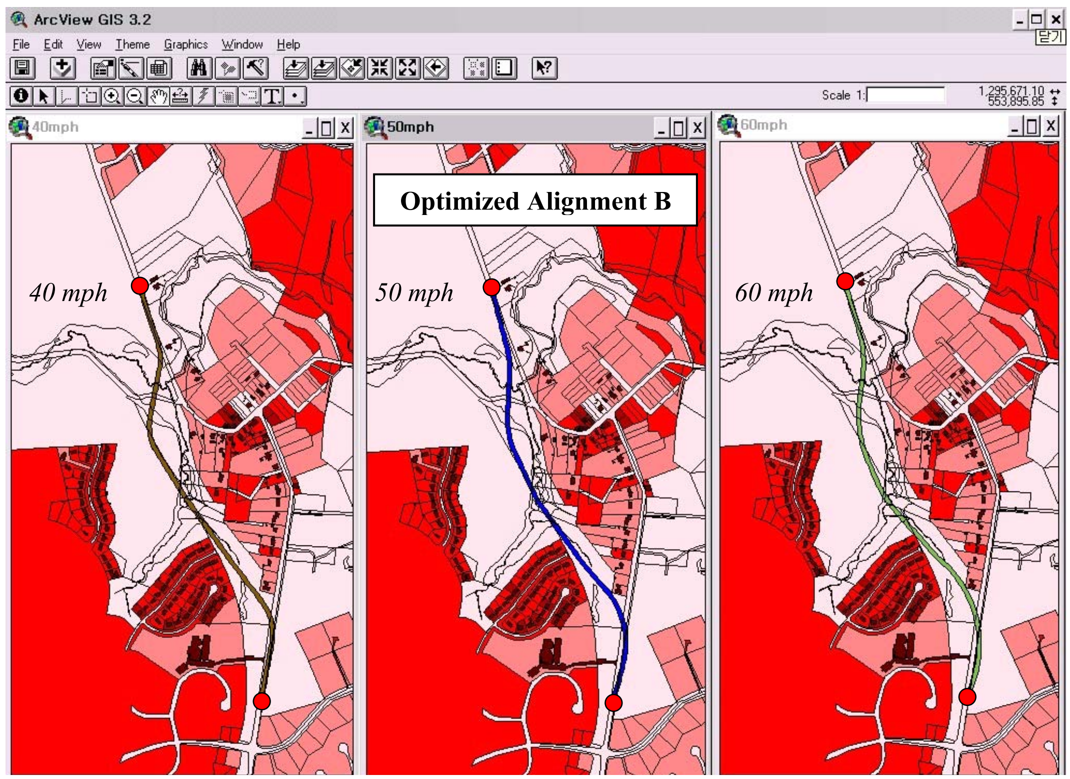
Figure 8.8(a2) shows the horizontal profile of the optimized alignment found with the four cost components (C_L+C_S+C_M+C_R) of the objective function (i.e., Case 2). As shown in the figure, this alignment hardly affects the expensive land areas and is relatively circuitous in avoiding the environmentally sensitive areas. However, its vertical alignment is still not optimized (i.e., it still has a huge difference with the ground elevation) since the model objective function of Case 2 does not consider the earthwork cost component (see Figure 8.8(b2)).

The horizontal and vertical profiles of the optimized alignment resulting when we consider all the five major costs (C_L+C_S+C_M+C_R+C_E) are presented in Figure 8.8(a3) and (b3), respectively. Although the horizontal profile of this resulting alignment (Case 3) is similar with that of Case 2, its vertical alignment is quite different. As shown in Figure 8.8(b3) its vertical profile closely follows the ground elevation. This is because horizontal and vertical alignments are optimized jointly while minimizing its earthwork cost as well as the other four cost components.

Note that the structure cost (C_S) and maintenance cost (C_M), although also dominating in the alignment construction, are less sensitive to the geometry of the alignment compared to the other components.

8.1.4.2 Sensitivity to Design Speed

This analysis tests the sensitivity of solution alignments to the design speed. The design speed is interrelated with many design features of a highway alignment (such as the horizontal curve radius, sight distance, transition curve length, and vertical curve length (crests and sags) of the alignment). In the model, it is specified by model users as an input, and the design features of the solution alignments are computed based on the AASHTO design standards (2001). As shown in Figure 8.9, the model creates smoother and longer horizontal curves at higher design speeds. Of course, the higher design speed also forces the model to generate smooth and long vertical curves. This indicates that the model performs correctly in creating highway alignments that satisfy the AASHTO standards.

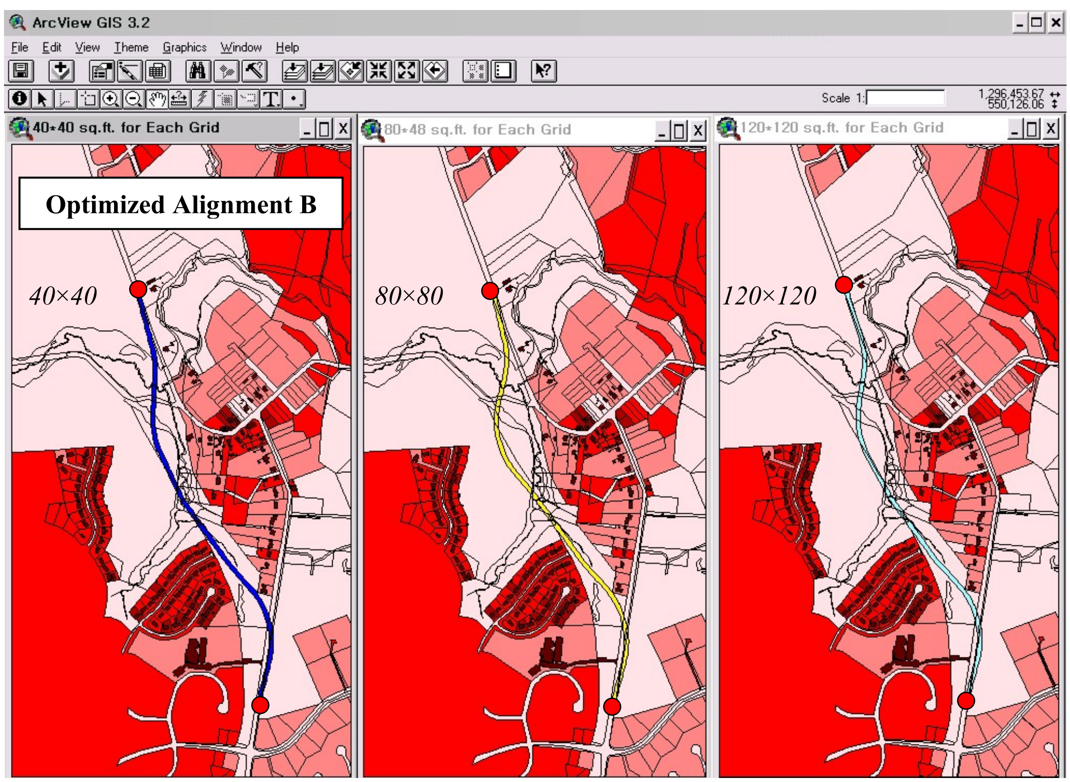


Design speed (mph)	Total agency costs ($)	Minimum curve radius (ft)	Environmental impact		Length (ft)	Computation time (hr)
			Type1 areas taken by alignments (ft2)	Residential relocation (No.)		
40	4,520,342	485	0	0	4,342.0	4.62
50	4,328,432	758	0	0	4,302.0	4.68
60	4,638,662	1,032	0	0	4,340.2	4.67

Figure 8.9 Sensitivity of Optimized Alignments to Design Speed

8.1.4.3 Sensitivity to Elevation Resolution

Resolution of the input ground elevation may also significantly affect the quality of solution alignments as well as model computation time. This may occur because the rough resolution of the ground elevation may decrease the accuracy of the earthwork cost estimation. As shown in Figure 8.10, there are striking differences in earthwork cost estimation between three optimized alignments generated with different input grid sizes even though they have very similar horizontal profiles; the earthwork cost significantly increases with rough grid size. This indicates that the model may produce unreliable earthwork estimates if the grid sizes are excessive, since terrain elevation estimates may then be too rough. Thus, a fine grid size is recommended in order to estimate the earthwork cost more precisely.

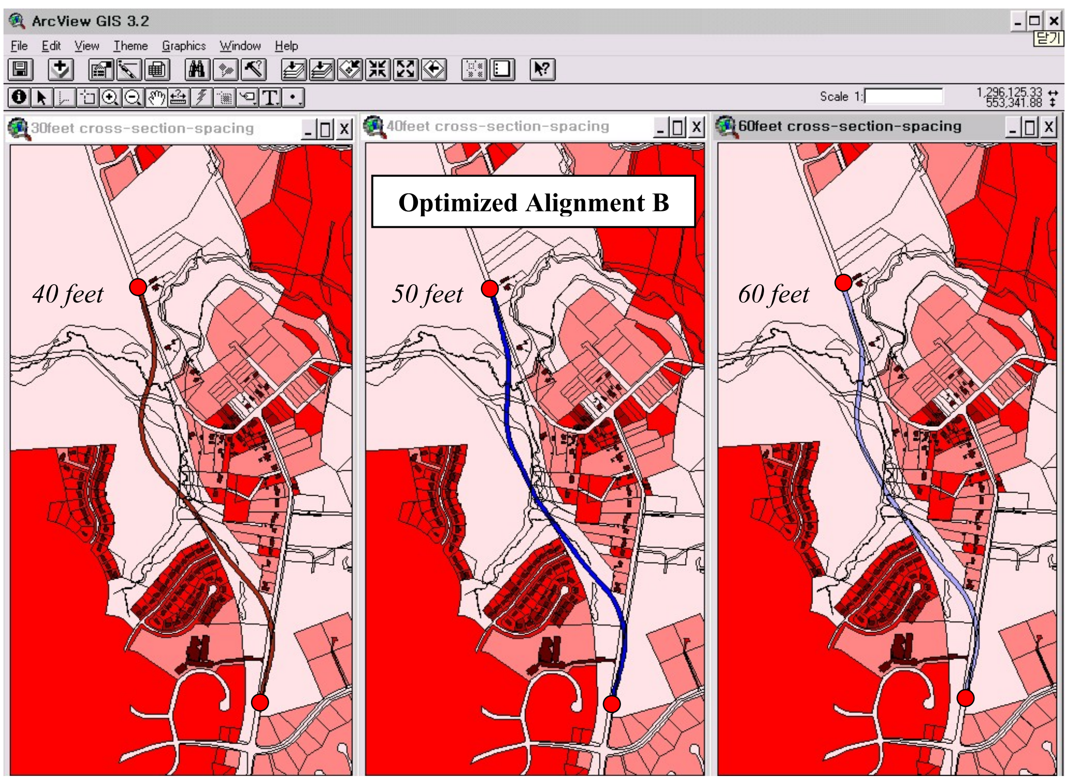


Unit grid size (ft×ft)	Total agency costs ($)	Earthwork cost ($)	Environmental impact		Length (ft)	Computation time (hr)
			Type 1 areas taken by alignments (ft^2)	Residential relocation (No.)		
40×40	4,328,432	1,599,586	0	0	4,302.0	4.68
80×80	5,876,282	2,809,691	0	0	4,369.0	4.63
120×120	6,014,216	3,195,195	0	0	4,331.4	4.50

Figure 8.10 Sensitivity of Optimized Alignments to Elevation Resolution

8.1.4.4 Sensitivity to Cross-Section Spacing

Figure 8.11 presents sensitivity to unit cross-section spacing, which is used as the earthwork computation unit of the model. It indicates that the earthwork cost and alignment length can vary depending on the unit cross-section spacing. Note that the cross-section spacing directly influences the precision of earthwork cost computations in the model. Moreover, the alignment length also is affected by the overall earthwork cost since the model seeks to reduce all the considered costs that are affected by the alignment length. In general, however, the variation of earthwork cost due to the differences of cross-section spacing is not significant.



Cross-section spacing (ft)	Total agency costs ($)	Earthwork cost ($)	Environmental impact		Length (ft)	Computation time (hr)
			Type 1 areas taken by alignments (ft^2)	Residential relocation (No.)		
40	4,672,390	1,638,947	0	0	4,390.9	4.77
50	4,328,432	1,599,586	0	0	4,302.0	4.68
60	4,407,257	1,613,784	0	0	4,319.4	4.64

Figure 8.11 Sensitivity of Optimized Alignments to Cross-Section Spacing

8.2 Case Study 2 (Maryland ICC Example)

In the second example, the alignment optimization model is applied to a major highway construction project in the State of Maryland, named the Intercounty Connector (ICC) project. Such a case study is designed for demonstrating the network optimization feature of the model proposed in Chapter 6.

This case study is organized as follows: After a brief description of the ICC problem in section 8.2.1, the next section presents input data employed in this model application. In section 8.2.3, optimized solutions found by the model are presented along with a goodness test. The sensitivity of solution alignments to analysis period is also presented in that section.

8.2.1 Problem Description

Overview of the ICC Study

The Intercounty Connector (ICC) has been proposed as a multi-modal transportation improvement to help address traffic needs between the I-270/I-370 and I-95/US-1 corridors within central and eastern Montgomery County and northwestern Prince George's County in the State of Maryland (See study area map in Figure 8.12). Many local, state and federal agencies as well as consultant companies have been working cooperatively to facilitate the progress and effectiveness of the ICC study. According to the Maryland Department of Transportation (MDOT, 1997), the need for the ICC is based on the following factors:

- "The I-270 corridor, which is one of the premier highway facilities providing direct cross-count routes in the State of Maryland, has only one access-controlled highway linking it to the I-95 Corridor. I-95 not only has extensive existing and planned development straddling it throughout the corridor between Washington and Baltimore, but also serves to connect the Washington Metropolitan Area to Baltimore and the entire northeast United States.

- The one access-controlled link connecting I-270 and I-95 is I-495 (the Capital Beltway), which is currently operating at capacity during peak periods, causing many persons traveling between the I-270 and I-95 corridors to utilize the local roadway system instead. These roads are not designed or intended to carry this longer distance travel. Furthermore, the Beltway is at the southern perimeter of the ICC study area and therefore does not provide a direct cross-county route for traffic in this area.
- Numerous roadways within the study area currently operate at or near capacity and have fairly high accident rates due to the many entrances and intersections.
- There is a lack of continuous east-west express transit service.
- The number of trips within the ICC study area, especially east-west trips, is expected to increase substantially in coming years.
- The number of intersections and roadway links in the ICC study area operating at or near capacity is also expected to increase substantially."

Given such needs, the purposes of the ICC are to:

- "Connect the existing and planned development areas between and adjacent to the two corridors with I-270 and I-95.

- Connect, in an environmentally responsible manner, the I-270 and I-95 corridors and accommodate, safely and efficiently, the east-west transportation movements between the corridors.
- Relieve congestion on existing roads not meant to accommodate cross-county traffic".

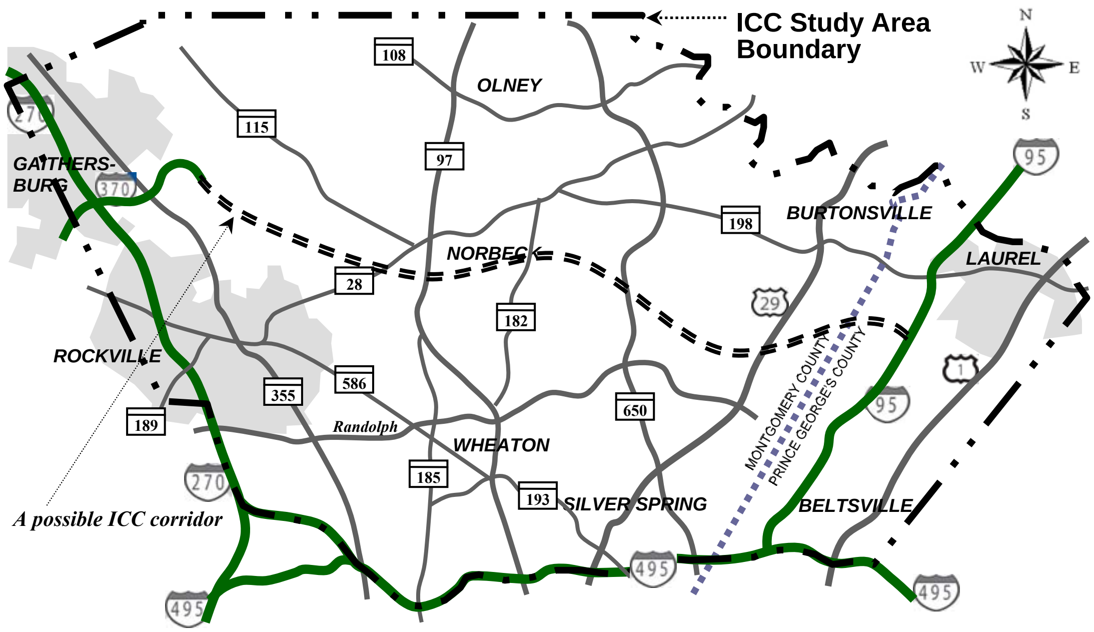


Figure 8.12 ICC Study Area Boundary

Description of Model Application to ICC Project

As stated above, the ICC study is a large-scale transportation improvement project in terms of time, space, and funding. Various critical factors (such as political, environmental, geographical and even capital investment issues) are interrelated, and vast amounts of data and resources are required for the problem. The highway alignment optimization model is also applied to this project in order to identify the best alternatives for the ICC; furthermore, the model's network level optimization capability is demonstrated through this case study. A problem description and the assumptions defined for this case study are presented below.

Problem description:

- In this case study, not only the highway alignments themselves but also their two endpoints and cross-points with existing roads are simultaneously optimized throughout the model application.
- Furthermore, traffic improvements due to the addition of the new alignments on the existing road network are also considered in the optimization process besides the other major alignment sensitive costs. Thus, the model objective function used in the ICC application is sum of (i) total user cost saving and (ii) total agency cost, as follows:

$$\Delta C_{User} + C_{T\_Agency} = (\Delta C_T + \Delta C_V) + (C_L + C_R + C_E + C_S + C_M).$$

Note that the accident cost (C_A), which is another component of the user cost, is suppressed from the model objective function in this application.

Assumptions and limitations:

- Only major highways (at least state level) are selected for specifying the existing road network, which is required for the traffic assignment process.
- Two continuous search ranges for the start and end points of new alignments are assumed to be known (along the I-370 and I-95, respectively), and a trumpet-type interchange is considered at each endpoint.

- Three major highways (MD-97, MD-650, and US-29) run between the two endpoints, and they are almost unavoidably intersected by the new alignments.
- For linking those major highways and new alignments, four different types of cross-structures are considered (see, below), and the best structure type of each cross-point is determined during the optimization process.
  - 4leg at-grade intersection
  - Grade separation
  - Clover interchange
  - Diamond interchange
- Traffic operating on the ICC study area in a base year (i.e., a base year O/D trip matrices) is known and increases annually with a given growth rate.
- A roughly digitized horizontal map is used here because preparation of a detailed GIS map is relatively quite expensive for model application to the large-scale project.

8.2.2 Input Data Preparation

Road Network

20 major highways are selected to represent an existing road network of the ICC study area (See Figure 8.12). These highways are employed to construct a network incidence matrix, which is used for an input of the traffic assignment process. Note that the incident matrix is kept updated during the optimization process if newly generated highways are added to the existing road network. Characteristics of the major highways, such as number of lanes, capacity, and speed limit are presented in Table 8.5.

Table 8.5 Characteristics of Major Highways in the ICC Study Area

Road name	No of lanes	Speed limit	Capacity per lane	Access Control*
I-95	8	65	2200	Full
I-270/I-495	12	60	2200	Full
I-495/I-370	8	60	2200	Full
US-29	6	55	2000	Partial
MD-198	4	55	2000	General
MD-183/MD-185/MD-193/MD-197/MD-198 MD-201/MD-355/MD-586/MD-650/MD-97	6	45	1800	General
MD-28/US-1	4	45	1800	General
MD-182/MD-189	4	40	1800	General
MD-198/MD-97	2	45	1800	General
MD-108	2	40	1800	General

* Full: fully access controlled highways without use of at-grade intersections; only interchanges and grade separations are used.
Partial: partially access control highways with mixed use of grade separations, interchanges, and at-grade intersections
General: No access controlled highways

Traffic Information

<u>*Zonal Descriptions*</u>

According to the Metropolitan Washington Council of Government (MWCOG), the Washington Metropolitan area is divided into 2,191 Traffic Analysis Zones (TAZ) consisting of parts of Maryland, Washington DC, and Northern Virginia. Among them, 423 TAZs, which are possible affected by the new ICC construction, are selected for the model application as shown in Figure 8.13. Note that 198 TAZs identified as the immediate ICC impact areas by the MDSHA (Clifton and Mahmassani, 2004) are included in the selected TAZs (See Table 8.6).

Table 8.6 Immediate ICC Impact Area by TAZ and Jurisdiction Boundary

County Name	Zone Number	No. of TAZs	State
Montgomery	394~468, 473~509, 526~556, 577~582, 585~592	157	MD
Prince Georges	781~792, 865~891	39	MD
Howard	1083	1	MD
Anne Arundel	1091	1	MD
Total	-	198	-

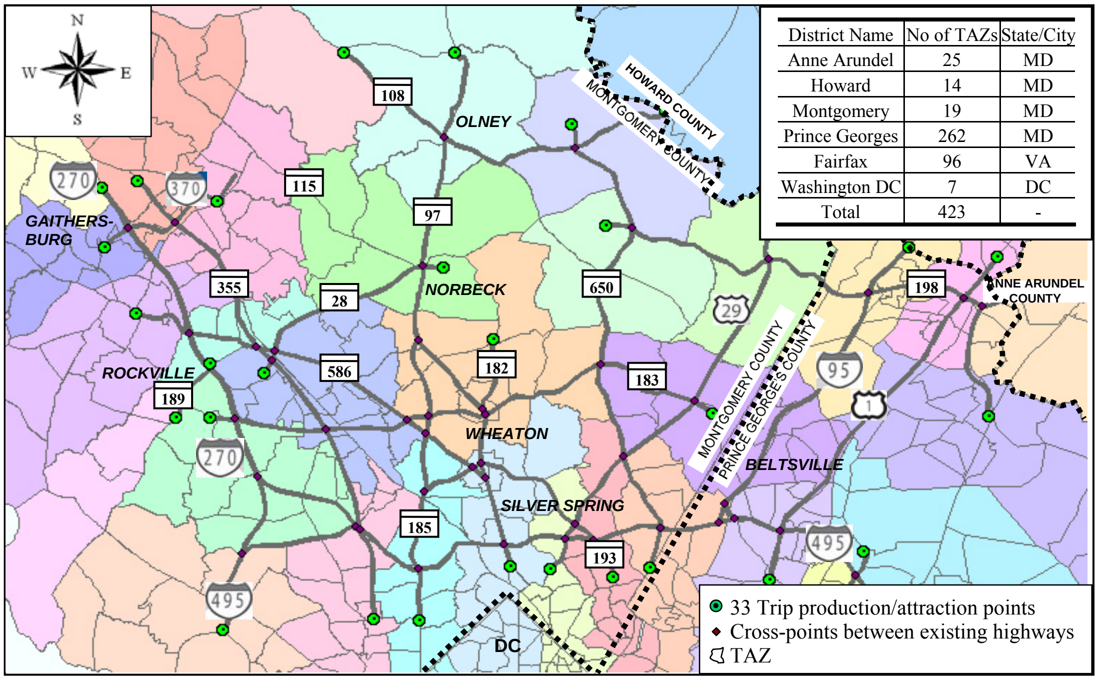


Figure 8.13 Selected TAZs for Model Application to ICC Project

<u>O/D Trip Matrices</u>

The year 2010 is assumed to be the base year for the model application to ICC study. Two types of modes (auto and truck), and three time periods (AM-peak, PM-peak, and OFF-peak) are considered. The base year O/D trip matrices for different modes and different time periods were obtained from the MWCOG. Note that O/D tables for different trip purposes (e.g., home-based work, home-based shopping, and non-home-based work trips) are not considered in this case study.

As shown in Figure 8.13, 33 trip production/attraction points (i.e., centroids) are heuristically identified (mostly) at the ends of the existing highways. These points are designed to aggregate O/D trip pairs between the selected TAZs. Each point represents several TAZs near from it (i.e., its corresponding TAZs are identified based on the distance from it). Thus, 178,929 (423×423) O/D pairs are aggregated to

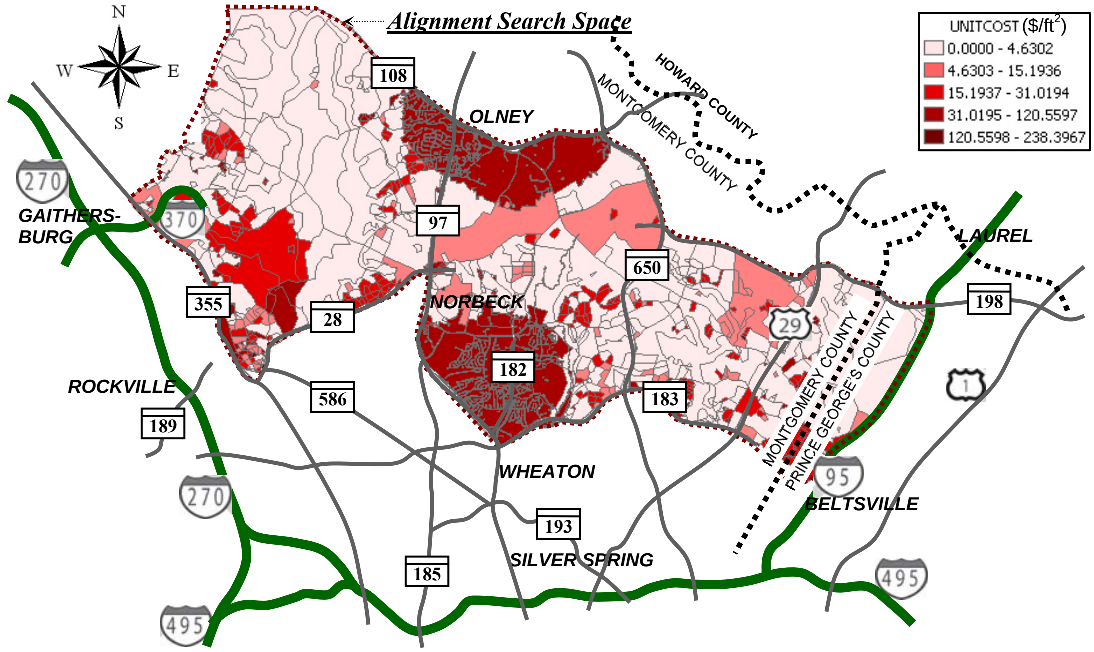
1,089 (33×33) pairs. The O/D trip matrices used in this case study are summarized in Appendix A.

GIS Map Preparation

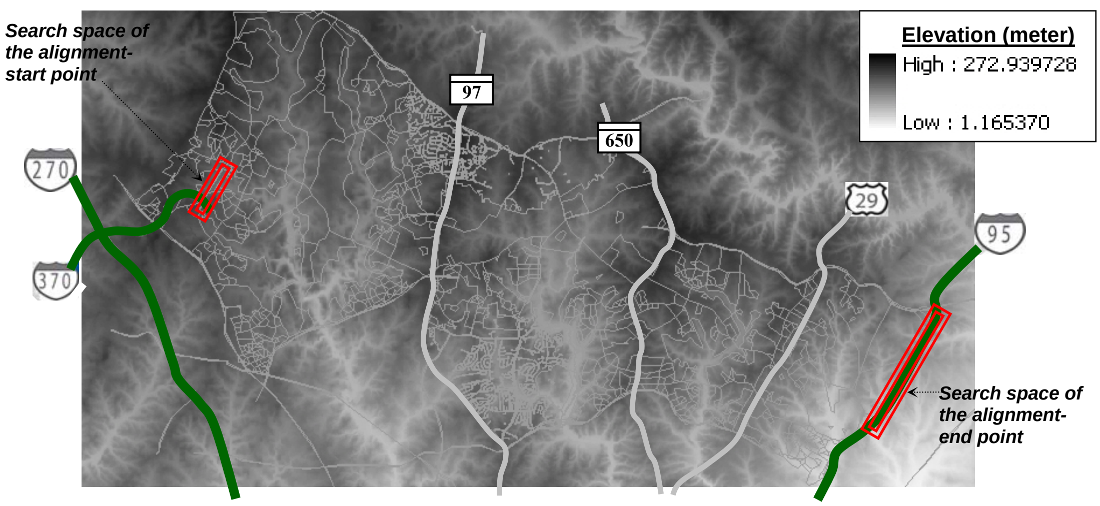
An alignment search space is specified within the ICC study area as shown in Figure 8.14. The total area size of the search space is about 108,362.4 acre (16.6 mile long and 10.2 mile wide). The search spaces for the start and end points of the new alignments are identified along the I-370 and I-95, respectively, as shown in the figure. The Euclidean distance between the start and end points is approximately 14.44 mile (23.243 km).

Horizontal Map

Through a horizontal map digitization process, more than 2,000 geographic entities (including rivers, parks, wetlands, existing highways, and residential and commercial properties) are represented as polygons; these retain their unique property information (such as, spatial location, area and property value). The unit cost ($\$/ft^2$) of each property in the search space is obtained from MDProperty View 2003. Note that the horizontal map of the search space (shown in Figure 8.14(a)) is somewhat more roughly digitized here rather than for the Brookeville case study because digitizing all detailed geographic entities (e.g., all individual building structures) in the large-scale project area is very expensive. Thus, environmental impact summaries and right-of-way costs of the alignments resulting from the model application to the ICC study may not be as accurate as those of the Brookeville case study.



(a) Profile of Unit Property Cost in the Selected Search Space



(b) Profile of Ground Elevation in the Selected Search Space

Figure 8.14 Alignment Search Space Selected for ICC Case Study

Ground Elevation Map

A DEM, which provides ground elevations of the ICC study area with a grid base, was downloaded from USGS website as shown in Figure 8.14(b). Note that the DEM is used to calculate the alignment earthwork cost in the model. In the DEM, the study area ground elevations are divided into evenly spaced grids of size 30meters×30meters (103feet×103feet). Finer grids may be selected for precise earthwork calculation as desired. The elevation range in the ICC study area is 4 to 895 feet (1 to 273 meters). The darker areas represent higher elevations.

Important Input Parameters

Input variables required for computing the alignment-sensitive costs in the model's application to the ICC study are summarized in Table 8.7. These are, for instance, road width and design speed as (i) agency cost variables, and annual traffic growth rate and truck percentage in the traffic as (ii) user cost variables.

Regarding the agency cost variables, design speed of the proposed alignment for the ICC is initially set to 60mph (96kph), and its cross-section is assumed to represent an 8-lane major highway with a 106 foot width (11 feet for lanes and 9 feet for shoulders). The distance between station points (i.e., cross-section spacing) of the new alignment is set to 50 feet (15 meters), and the minimum vertical clearance required for crossing with existing highways is assumed to be 15 feet (4.5 meters).

Regarding the user cost variables, the annual traffic growth rate and truck percentage in the traffic of the ICC study area are assumed to be 10% and be 15%, respectively. In addition, the interest rate and analysis period are set to 3% and 5 years, respectively.

Unit travel time value ($/hr), average vehicle occupancy (person/veh), and fuel prices ($/gallon) for autos and trucks are presented in Tables 7.5, 7.6, and 7.8, of Chapter 7, respectively. Please refer to Table 8.7 for the other important input parameters used in the ICC case study. Note that values of all these input variables should be cautiously defined because they may sensitively affect the resulting alignments.

Table 8.7 Baseline Inputs Used in the Model Application to ICC Case Study

	Input variables	Value	
Agency cost variables	No. of intersection points (PI's)	8 ~ 12	
	Road width	106 foot, 8-lane road (11" lane, 9" shoulder)	
	Design speed	60 mph (96 kph)	
	Maximum superelevation	0.06	
	Maximum allowable grade	5 %	
	Coefficient of side friction	0.16	
	Longitudinal friction coefficient	0.28	
	Distance between station points	50 feet (15 meters)	
	Fill slope	0.4	
	Cut slope	0.5	
	Earth shrinkage factor	0.9	
	Unit cut cost	35 $\$/\text{yard}^3$ (45.5 $/m3)	
	Unit fill cost	20 $\$/\text{yard}^3$ (26 $/m3)	
	Cost of moving earth from a borrow pit	2 $\$/\text{yard}^3$ (2.6 $/m3)	
	Cost of moving earth to a fill	3 $\$/\text{yard}^3$ (3.9 $/m3)	
	Unit length-dependent cost	400 $/feet (656 $/meter)	
	Terrain height ranges	4~ 895 feet (1 ~ 273 meters)	
	Unit land value in the study area	0 ~ 238 $/ft2 (0 ~ 2,562 $/m2)	
	Structure type on the start & end points	Trumpet interchanges	
	Structure types on the cross-points with existing highways	Grade separation, 4-leg at-grade intersection, Clover and Diamond interchanges	
User cost variables	Traffic growth rate	5%	
	Truck percentage in the traffic	5%	
	Interest rate	3 %	
	Analysis period	5 years	
	Base year O/D	2010 O/D trip matrices (trips/hr)	(see Tables A.1 ~ A.4)
	Unit travel time value	9.28 $/hr for auto drivers 16.84 $/hr for truck drivers	(See Table 7.5)
	Average vehicle occupancy	1.550 persons/auto 1.144 persons/truck	(See, Table 7.6)
	Fuel prices	2.73 $/gallon for auto 2.67 $/gallon for truck	(See Table 7.8)
	Number of major highways used for constructing an existing road network	20 (at least state level)	
	Number of centroids (trip production/ attraction points) pairs	1,089 (=33×33)	

8.2.3 Optimization Results

8.2.3.1 Determination of Traffic Reassignments

Figure 8.15 shows example solution alignments possibly generated at the beginning of the model search process (called initial population stage). Such initial population members may include straight alignments as well as some possible candidate alignments selected based on judgments of highway designers and planners. The solutions are improved over successive generations with the aid of the customized genetic operators (see section 5.2.1 and Jong, 1998) and the efficient solution search methods (proposed in Chapters 3 and 4 for FG and P&R approaches, respectively) after the initial population stage is completed.

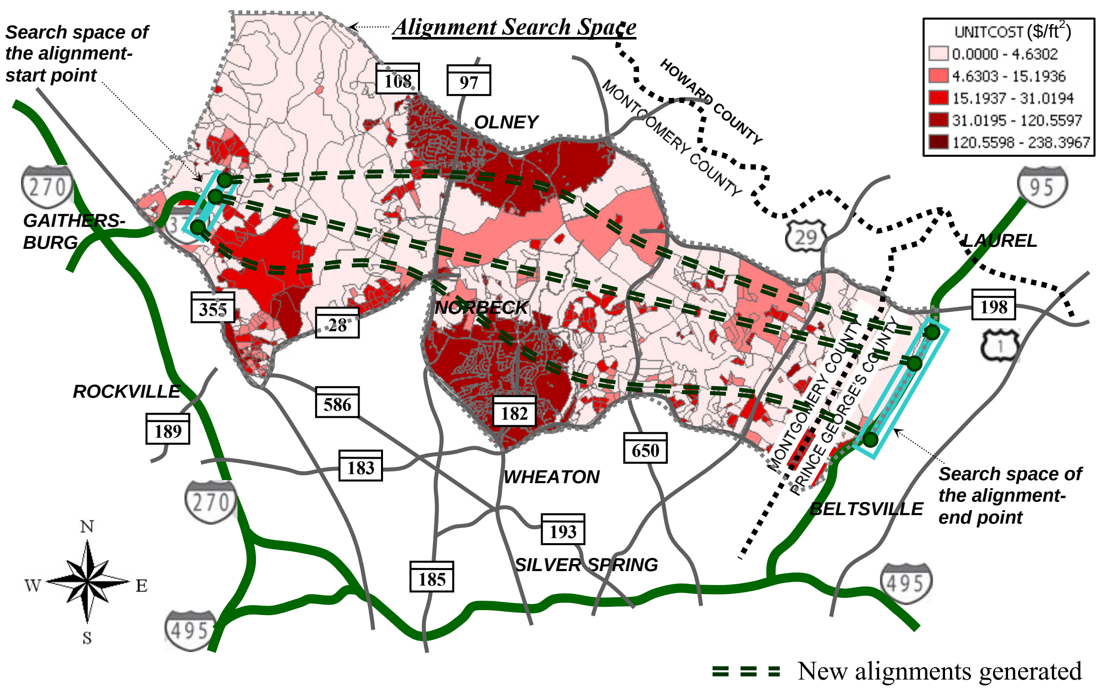


Figure 8.15 Example Alignments Possibly Included in the Initial Population of the ICC Case Study

Recall that the bi-level optimization feature of the model (see section. 6.2) is designed (i) to update the configuration of the road network after a new highway alignment is generated, and next (ii) to find equilibrium traffic flows in the updated network from the traffic assignment process, and (iii) finally to evaluate the total cost of the new highway (including the user cost savings as well as agency cost) associated with its construction.

It should be noted, however, that the bi-level optimization may not be efficient in cases when the assignment results for the networks updated with different highway alternatives are very similar. For instance, the difference in traffic volumes which would operate on the new highways shown in Figure 8.15 may be negligible although their start and end points as well as horizontal (and even vertical) alignments generated from the model significantly differ. In such a case, processing the traffic assignment (i.e., finding equilibrium traffic flows) for every updated network with the new alternative generated is wasteful. Procedure 8.2 is developed here for determining whether the bi-level optimization feature is needed during the optimization procedure for given problems. Note that this procedure is preprocessed with sample solutions generated at early stages of the alignment optimization including initial population.

Preprocessed Traffic Assignments (8.2)

STEP 1: Generate initial population, including straight and curved alignments.

- STEP 1-1: Identify domains of highway endpoints specified for the endpoint generations.
  - As shown in Figure 8.16 (a), domains of the highway start and end points are divided into 3 road segments each in the ICC case study (i.e., n_{seg1} =3, n_{seg2}=3).

- STEP 1-2: With each pair of road segments, generate sample alignments including straight and curved alignments.
  - In the ICC case study, totally 9 (=3×3) segment pairs are identified for the endpoints generation, and at least 5 sample alignments are generated with each segment pair. Thus, more than 45 (=5×9) highway alternatives are generated during the initial population stage (see Figure 8.16 (b)).

STEP 2: Find x_{new}^{j} for all j (j=1, …, N_{ipop}) from the traffic assignment process

, where x_{new}^{j} = predicted traffic volumes that would operate on the j^{th} new highway of the initial population, and can be found through the traffic assignment process for the network updated with the new highway addition; N_{ipop}= total number of sample highways generated in the initial population ($N_{ipop} \geq 5 \times n_{seg1} \times n_{seg2}$).

STEP 3: Compute x_{new_M} and x_{new_SD}

, where x_{new_M} and x_{new_SD} = mean and standard deviation of all x_{new}^{j} (j=1, …, N_{ipop}), respectively.

STEP 4: Compute x_{new_CV}

, where x_{new_CV} = coefficient of variation[20] of the predicted traffic volumes (x_{new}^{j}) operating on the initial population members; $x_{new\_CV} = x_{new\_SD} / x_{new\_M}$

[20] A small coefficient of variation indicates that the assignment results for different alternatives are relatively consistent.

STEP 5: Check whether $x_{new\_CV} \le F_{TA}$ or $x_{new\_CV} > F_{TA}$

, where F_{TA} = a user-specifiable threshold value for determining whether the bi-level optimization feature is needed. (Note that we assume F_{TA}=0.05 in the ICC case study.)

- If $x_{new\_CV} \le F_{TA}$ (i.e., the traffic assignment results are relatively consistent for the initial population):
  → Stop the traffic reassignment procedure for alternatives generated over the successive generations. Instead, the results of the preprocessed traffic assignments with the initial population will be used for estimating the user costs of those solutions.

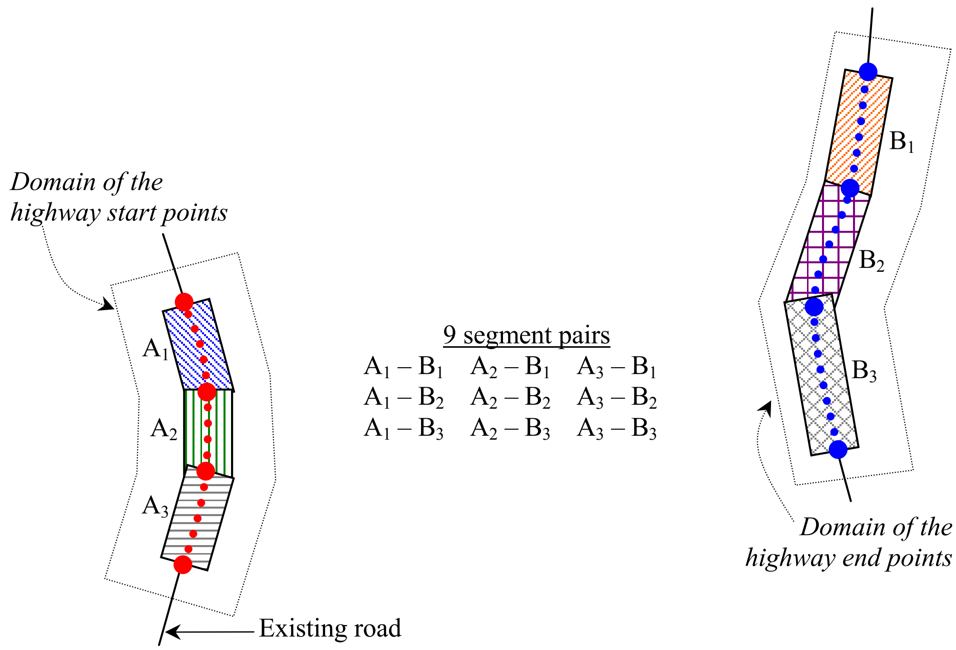
- Otherwise (if $x_{new\_CV} > F_{TA}$):
  → Keep processing the traffic reassignments (beyond the initial population stage) with additional alternatives generated until K_{TA}th generation (see STEP 6). Note that K_{TA} is a user-specifiable parameter, which is set to 50 generations in the ICC case study.

STEP 6: During the K_{TA} generations, compute $x^i_{new\_CV}$ for alternatives generated with the specified road segment pairs ($i=1,\ldots, n_{seg1} \times n_{seg2}$).

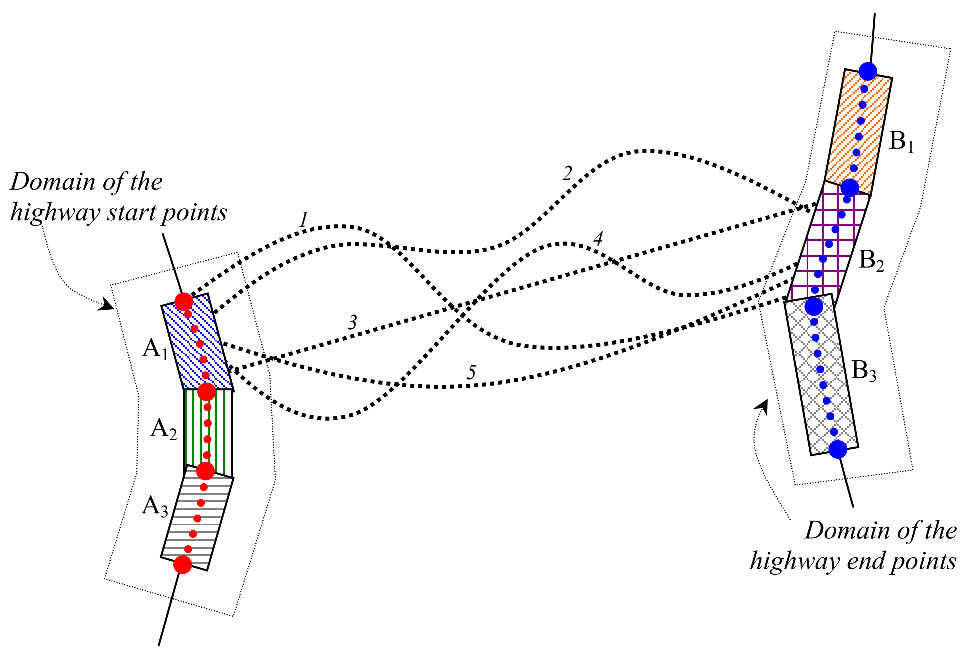
- Recall that 9 ($=n_{seg1} \times n_{seg2}$) pairs of road segments are specified in the ICC case study (see Figure 5.16 (a)), and the traffic reassignment results are saved for each segment pair during the K_{TA} generations:

  → Compute $\begin{matrix} x^1_{new\_CV} \\ \vdots \\ x^9_{new\_CV} \end{matrix}$, and

  → Check whether $x^i_{new\_CV} \le F_{TA}$ or $x^i_{new\_CV} > F_{TA}$ for $\forall\ i$
  - If $x^i_{new\_CV} \le F_{TA}$:
    The traffic reassignment results will be used for estimating the user cost of other alternatives generated with the corresponding road segment pair during the rest of generations.
  - Otherwise ($x^i_{new\_CV} > F_{TA}$):
    The traffic reassignment will be processed for all alternatives generated over the successive generations.



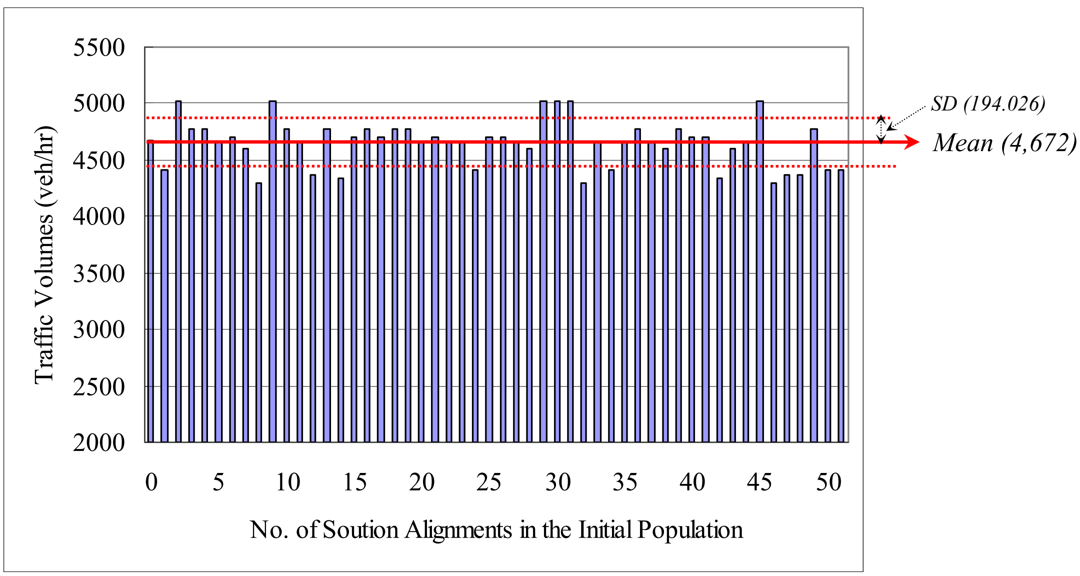
(a) Disaggregate Domains for Highway Endpoints Generation



(b) Example Highways Generated with Road Segment Pair A_1–B_2

Figure 8.16 Road Segment Pairs Specified for Preprocessed Traffic Assignment

Figure 8.17 shows the predicted traffic volumes (veh/hr) which would operate on the sample ICC alternatives generated at the initial population stage. About 50 alternatives are generated in the initial population of the ICC case study. This result indicates that the traffic volumes operating on the various alternatives are relatively similar (with x_{new_CV} =0.0415) despite their different start and end points locations and different horizontal profiles. Thus, the traffic reassignments are not performed through the successive model search processes (after the initial population stage); instead, the preprocessed assignment results are used for estimating the user costs of the alternatives generated during the rest of the search process.



# of lanes*	Mean (x_{new_M})**	Standard deviation (x_{new_SD})	Coefficient of variation (x_{new_CV})
8	4,672	194.0258	0.0415

* The proposed alignment is assumed to be a 106 foot wide, 8-lane road (refer to Table 8.7)

** Average hourly traffic (average of AM, PM, OFF-peak volumes) operating on the new alignments

Figure 8.17 Predicted Traffic Volumes Operating on the New Alignments of the Initial Population for the ICC Case Study

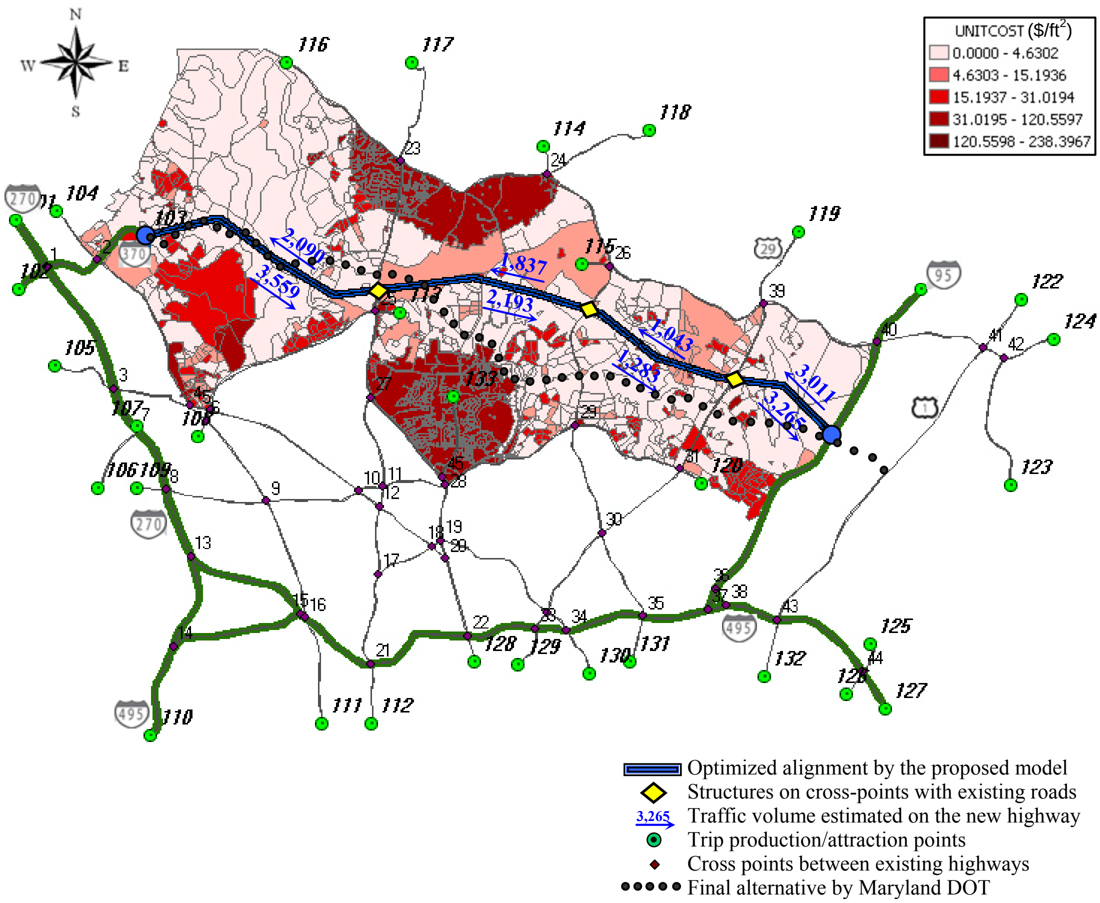
8.2.3.2 Optimized Alignments

The alignment optimization model searches over 300 generations (including the initial population stage) to find the cost effective ICC alternatives given the input data shown in Table 8.7. A desktop PC, Pentium Dual CPU (3.0 GHZ, 3.0 GHZ) with 2 GB RAM is employed to run the model, and about 8,400 alignments are evaluated during the search process. It takes a relatively long time (about 24 hours) to run through 300 generations because the ICC study area is very large (16.6 mile long and 10.2 mile wide) and contains many geographic entities.

Note that the model runs 5 times (searching over 300 generations per each) with different input PI's (8 to 12 PI's). As a result, the optimized solution found with 10 PI's seems the most preferable because of its lowest objective function value, although alignment profiles and objective function values of all the five solutions are very similar. Figure 8.18 shows horizontal and vertical profiles of the optimized alignment obtained after 300 generations with 10 PI's (i.e., the most preferable one). As shown in the figure, the optimized alignment has seven horizontal curves that satisfy the given design standards, while avoiding the predefined control areas and high cost properties. In addition, its vertical alignment closely follows the ground elevation, while minimizing its earthwork cost. The new highway is 16.02 miles long, and three clover interchanges and two trumpet interchanges are built for facilitating turning movements at the crossing points with existing roads.

It is important to note here that the resulting alignment is found based on the model inputs provided in Table 8.7; thus, some limitations in data accuracy may exist. The resulting alignment may be further improved or changed if more precise and detailed inputs (such as more detailed environmental consideration and O/D traffic information) are provided. For readers' information, the ICC alternative finally proposed by the Maryland Department of Transportation is also shown in Figure 8.18.

(a) Horizontal Profile



(b) Vertical Profile

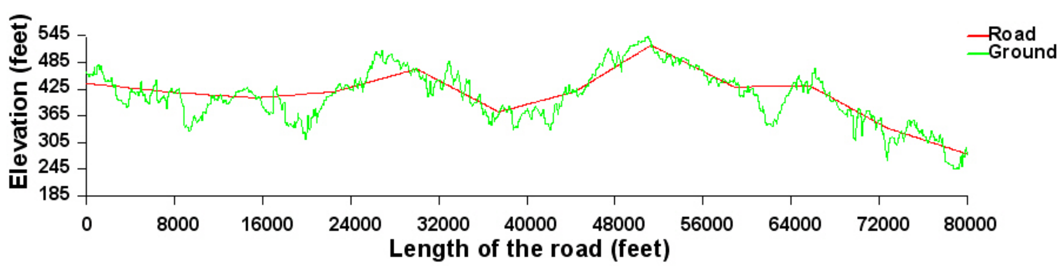


Figure 8.18 Horizontal and Vertical Profiles of Optimized Alignment for ICC Study

Figure 8.18 also presents average traffic volumes (veh/hr) which would operate on the new alignment in the base year (2010). These results are calculated from the traffic assignment process of the highway network updated with the new highway construction. Expected traffic improvements on three existing major highways (I-95, I-495 and I-495) due to the new highway construction are summarized in Table 8.8. The results indicate that traffic condition of the I-95 can be significantly improved after the system development (with 26% traffic reduction). Traffic on I-495 and I-270 can also be improved (with 18% and 8% reduction, on average) with the aid of the highway construction. Note that the input O/D trip matrices used for the assignment are presented in Appendix A.

Table 8.8 Average Traffic on Major Inter-State Highways before and after the New Alignment Construction (2010 base year)

Major Highways		Traffic Volume (veh/hr)		Reduction (%)
		No Build	Build	
Optimized Alignment	West Bound	-	2,575	-
	East Bound	-	1,995	-
I-95	South Bound	5,858	4,349	26
	North Bound	6,125	4,549	26
I-495	West Bound	6,050	4,908	19
	East Bound	5,749	4,767	17
I-270	South Bound	7,966	7,406	7
	North Bound	7,349	6,721	9

Change in Objective Function Value over Successive Generations

In order to assess the behavior of the objective function over the successive generations, the objective function values are plotted at various generations, as shown in Figure 8.19. It is observed that the objective function values in the first few generations are extremely high. However, the value drops considerably until about 79 generations. The improvement in the objective function value becomes very slow (almost negligible) after that. The final objective function value is about 16 million, which is reached at the 160th generation.

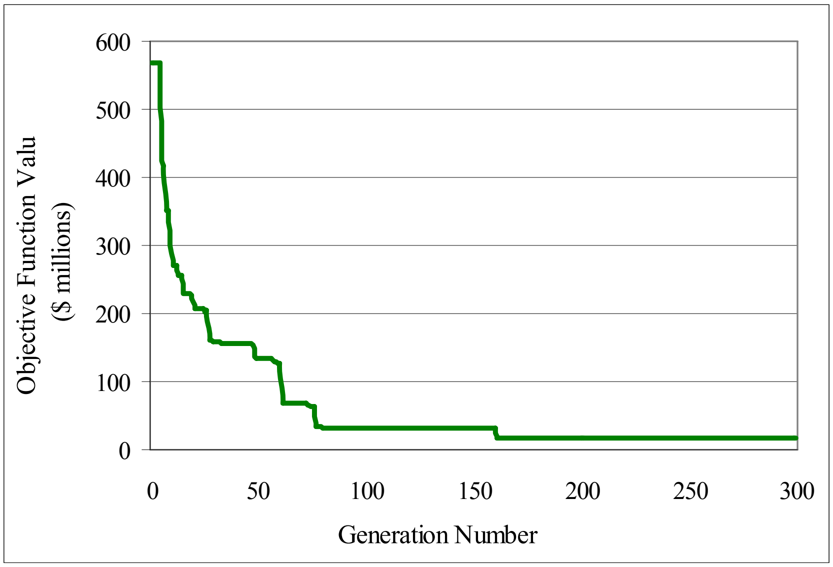


Figure 8.19 Changes in Objective Function Value over Successive Generations for ICC Case Study

Fraction of Various Costs

It is noted that since the objective function consists of the seven cost components (travel time, vehicle operation, earthwork, right-of-way, length-dependent, structure, and maintenance costs), the proposed optimization model attempts to find the best trade-off between the various cost components and obtain the minimum total while satisfying the specified geographical and design constraints. An analysis is performed to investigate the percentage of various costs in the model objective function value for the optimized alignment found for the ICC case study. A detailed breakdown of its objective function value is shown in Table 8.9, and Figure 8.20 shows the fractions of the various costs.

Table 8.9 Breakdown of the Objective Function Value of the Optimized Alignment for the ICC Case Study

Type of Cost		Millions ($)	Percentage (%)
Total agency costs	Earthwork	266.29	59.84
	Length-dependent	33.83	7.60
	Right-of-way	10.92	2.45
	Structures	131.08	29.46
	Maintenance	2.87	0.65
Subtotal		449.99	100.00
Total user cost savings	Travel time	-296.78	69.18
	Vehicle operation	-132.19	30.82
Subtotal		-428.98	100.00
Total costs (Objective function value)		16.08	

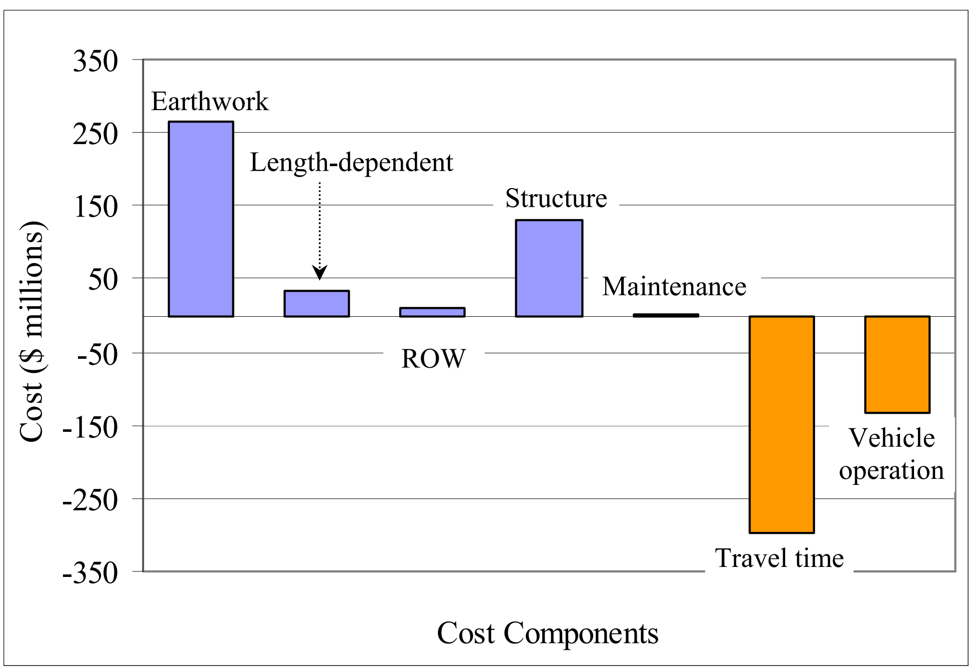


Figure 8.20 Comparison of Various Costs for Optimized Alignment of ICC Study

The results indicate that (i) travel time cost saving, which can be obtained from the system improvement and (ii) the earthwork cost required for the new road construction, make up the first and second highest fractions of the total objective function value, respectively. They dominate the other cost components included in the objective function. The vehicle operating cost saving and the structure cost also account for large fractions of the total objective function value. These results suggest that care should be taken in using appropriate cost functions in the optimization model to reflect all important costs, although most highway agencies in the field tend to ignore the user costs in the road planning phases. Note that the negative values of the user cost savings indicate that the user costs estimated before the system improvement are greater than those after the road construction.

The impacts of the optimized alignment on environmentally sensitive regions of the ICC study area are not presented here since the input land-use maps (which provide spatial locations of various environmentally important features of the study area) used in the ICC case study are not detailed and precise enough.

Sensitivity to Analysis Period

A sensitivity analysis is performed to observe when the user cost savings due to the new highway development exceed the total agency cost required for its initial construction and periodical maintenance. The variation of the total cost (i.e., objective function value) with respect to different analysis periods are presented in Table 8.10. The result indicates that 5 years after the new road construction the user cost savings exceed the total agency cost so that the total cost becomes negative value, which means that the highway development project starts to benefit from the year 2016.

Table 8.10 Sensitivity of Objective Function Value to Analysis Period

Unit: ($) millions as of 2010 base year

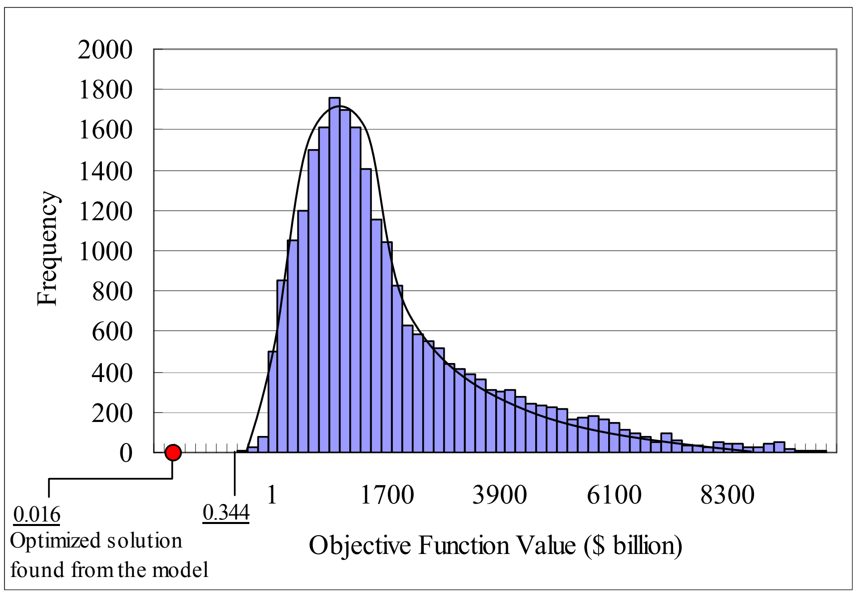
Analysis Period	Total Cost	Total Agency Cost	Total User Cost Saving
1	360.27	442.67	-82.40
2	276.81	443.27	-166.46
3	191.64	443.86	-252.22
4	104.72	444.43	-339.72
5	16.01	444.99	-428.98
6	-74.51	445.53	-520.04
7	-166.90	446.05	-612.95
8	-261.17	446.56	-707.73
9	-357.37	447.05	-804.42
10	-455.55	447.53	-903.07

8.2.3.3 Goodness Test

Recall that the ICC case study is a quite different model application compared to the Brookeville example. Highway endpoints as well as its alignments are simultaneously optimized, and traffic improvements on the existing road network due to a new highway development are considered together with various highway agency costs in the ICC application. Thus, a statistical analysis is also performed here to test the goodness of the best solution found by the model.

A set of sample solutions (30,000) is randomly generated to compare them with the optimized solution found by the model. It is observed that the best solution of the random sample yields an objective function value 344 million, while the objective function value of the worst one is 9,955 billion. The sample mean is about 2,065 billion and the standard deviation is 2,113 billion. A distribution diagram of the random sample and its descriptive statistics are presented in Figure 8.21. The relative position of the optimized solution found is also indicated on that figure. The results show that the sample distribution has an offset of 344 million, which is much higher than the optimized solution (16 million) found by the model. This means that the optimized solution dominates all the sample solutions; it is 21 times smaller than the best of 30,000 randomly generated solutions. Such results give us confidence that the optimized solutions found by the model are very excellent when compared to other possible solutions to the problem.

(a) Distribution of Objective Function Values



(b) Descriptive Statistics of Objective Function Values (unit: $ billion)

	Min	Max	Mean	Median	Standard Deviation
Random Search	0.344	9955	2065	1390	2113
Model	0.016				

Figure 8.21 Comparison of Solutions Found from Random Search and Optimization Model for the ICC Case Study

Chapter 9: Conclusion and Future Work

The selection of highway alternatives (including their geometric design, cost-benefit analysis, and analysis of their impacts to the environmental system) is a very complex and challenging problem due to the large number of conflicting factors that must be resolved, the great amount and variety of information that must be compiled and processed, and the numerous evaluations that must be performed. The process of evaluating even one candidate alignment with existing methods is so expensive and time consuming, that typical studies can only afford to evaluate very few alternative alignments. Several mathematical highway design models have been developed to reduce time, cost, and errors of the highway design process. However, due to the difficulties of the problem, a limited number of previous models can yield theoretically reliable and practically useful results, and thus none are widely adopted for design highway alignments in real world applications[21].

In this dissertation, we examine the properties of alignment optimization problems, and review all models found in the literature. The weak points of existing models and the directions of improvements are identified. Based on this work, a comprehensive highway design model which thoroughly describes the complex alignment optimization problem is developed in the preceding chapters. In this chapter, we summarize the main findings and contributions of the dissertation. The

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[21] Quantm (http://www.quantm.net/index.cfm), a highway design software seems to be used in some real world projects; however, its theoretical background has not been discussed in public.

recommendations for future research to address current model limitations and challenges are also presented in this chapter

9.1 Summary

Through this dissertation, we seek realistic three-dimensional (3D) highway alignments that best improve the existing highway system, while considering their geometric designs, various costs associated with construction, and even environmental impacts to the study area. In response, a state-of-the art model for optimizing highway alignments is developed. The proposed model can simultaneously optimize (i) highway alignments (horizontally and vertically) as well as (ii) their junction points (including its endpoints and intersection points with existing roads); furthermore, an equilibrium traffic assignment process is incorporated in the model framework to evaluate the traffic improvements due to the new highway addition to the existing road network. The assignment results are used for evaluating the traffic impacts of the alignment alternatives as well as the agency costs required for their construction. In addition, since the new highway construction may significantly affect environmentally sensitive areas (such as wetlands and historic areas) and human activities of the existing land-use system (i.e., residential and commercial areas), these factors are also accounted for in the alignment optimization process. The performance of the proposed model is well described through the application to real highway projects presented in Chapter 8, and its major capabilities are listed as follows:

- It generates realistic 3D highway alignments
- It can simultaneously optimize highway alignments as well as their junction points with existing roads.
- It considers complex geographical constraints based on user preferences beyond highway design constraints.
- It can evaluate detailed environmental impacts of the candidate alignments during the optimization process.
- It can evaluate traffic impacts of the candidate alignments on the existing road network during the optimization process.
- It can evaluate various agency costs required for highway construction during the optimization process.
- It can find optimized alignments reasonably fast using efficient solution search methods proposed in this work.
- It can help highway system operators (e.g., highway agencies), who are in charge of highway planning and design, to consider incomparably more design alternatives and variations than can be presently afforded.

9.2 Research Contributions

The main purpose of the proposed optimization model is to assist highway planners and designers in identifying promising alignments and evaluating them when considering a new highway construction to an existing road network. It is expected that they will greatly benefit from the proposed model, which offers well optimized candidate alternatives developed with automated GIS data extraction and comprehensive evaluation procedures, rather than merely satisfactory alternatives, in the planning stages of new highways. Furthermore, it can greatly reduce the resources (money and time) required for the traditional highway design process. The main contributions of this research are described as follows:

1. Considering Most Relevant Highway Evaluation Criteria for Optimizing Highway Alignments

One of the most important capabilities that an effective alignment optimization model should possess is to consider comprehensive lists of evaluation criteria in the optimization process. Various costs relevant to highways construction as well as their impacts to the existing highway system should be comprehensively evaluated for all alternatives considered in the model. However, although some previous models dealing with the highway design process may generate realistic highway alignments, they only consider a limited number of highway costs and even oversimplify the costs estimation. Traffic impacts of the new highways on the existing road network and their detailed environmental impacts are not considered in the previous models.

Models proposed to deal with the discrete network design problem (DNDP) may evaluate the traffic impacts of highway alternatives; however, they are impractical to use directly in a real highway construction project. Such macro-level models do not consider many significant factors to be considered in the highway design problem, such as geometric design features and environmental impacts of the new highways. Furthermore, they cannot generate realistic 3D highway alignments because highways and road junction points are represented with single lines and nodes, respectively, in the DNDP models.

The alignment optimization model proposed in this dissertation tries to realistically represent most relevant issues arising in the real highway construction project; furthermore, we take into account the advantages of the previous highway models in developing it. The highway evaluation criteria which are newly added and/or updated in the proposed model are summarized as follows:

- Costs associated with highway construction and management
    - Periodical highway alignment and bridge maintenance costs
    - Highway bridge costs
    - 3-leg structures costs for the endpoints of the new highway crossing with existing roads (e.g., trumpet interchanges, roundabouts, and at-grade intersections (3-legs))

Note that highway earthwork, right-of-way (land acquisition), and length-dependent cost functions are adopted (without further modification) from the previous HAO model developed by Jong and Schonfeld (2003) and Jha and Schonfeld (2000).

- Costs associated with highway impacts to the existing system
    - Traffic impacts to the existing and future highway users
        - Travel time savings from the highway development
        - Vehicle operation cost savings
        - Accident cost savings
    - Detailed environmental impacts to the land-use system
        - Number of property relocations required

- Area affected by new alignments (to historic places, wetlands, floodplains, streams, parklands, existing roads, and residential and commercial areas)

2. Bi-Level Model Framework for Optimizing Highway Alignments

A bi-level model framework, which is also used in the DNDP, is adopted in this dissertation for comprehensively optimizing highway alignments. The upper-level problem of the model structure is the highway alignment optimization (HAO) problem, which simultaneously optimizes 3D highway alignments and their junction points with existing roads, and the lower-level problem is the traffic assignment problem, which finds the traffic impacts of the new highways to the existing road network. By proposing the bi-level programming structure, the capability of the alignment optimization model expands to handle the network level problem. The proposed model can now evaluate the traffic impacts of the new highway alternatives on the existing road network during the optimization process beyond evaluating initial costs and constraints required for their construction.

3. Efficient Alignment Search Process

Two efficient constraint handling methods are proposed for maintaining feasibility of solutions generated from the integrated GAs and GIS-based alignment optimization model. Theses are Feasible Gate (FG) and Prescreening & Repairing (P&R) methods. In the model the FG methods are implemented for efficiently handling the solution alignments that violate geographical constraints; on the other hand, the P&R method is used for dealing with those alignments that violate highway design constraints. The concepts of these methods are as follows:

- FG methods
    - Realistically represent complex user preferences, environmentally sensitive areas, and gradient constraints of the alignment optimization problem so as to maximize the chance that alignments satisfying the restricted constraints are generated.
- P&R method
    - Repair (before the very detailed alignment evaluation) any candidate alignments whose violations of design constraints can be fixed with reasonable modifications. However, if violations of such constraints are too severe to repair, the infeasible alignments are prescreened before any detailed evaluation procedure.

Significant contributions of the proposed methods (to computation efficiency and to solution quality of the optimization process) are demonstrated through the model application to a real highway project. As a result, it has been shown that the model computation time is reduced by approximately 28% with the FG method, and its solution quality is improved throughout the search process (refer to section 3.4). This indicates that the FG method successfully assists the model in narrowing its horizontal and vertical feasible bounds based on the specified conditions including user preferences, and thus it can focus sooner on refining the feasible alignments and provide the optimized solutions much faster.

By incorporating the P&R method, it has been shown that the model can find optimized solutions with 23% computation time savings; furthermore, about 70% more solutions are considered during the optimization process compared to those without the method. This can be interpreted that the model can now avoid evaluating the infeasible alignments with its prescreening process and focus on refining feasible alignments with its repairing process.

It is importantly noted that such model improvements due to the proposed methods can significantly increase if the scale of the road project is enlarged (e.g., if the size of alignment search space is enlarged and/or if the number of geographic entities in the study area increases). The concepts of those methods may also be applied to many other complex optimization problems for computational efficiency.

4. Optimizing Highway Junction Points as well as Alignments

In the proposed model, besides the alignment of a new highway, its two endpoints as well as multiple intersection points with existing roads (if it crosses the roads) are simultaneously optimized. Such work is implemented in the model in order to represent the variation of highway users' route choice with respect to different highway junction locations. Furthermore, not only the highway junction locations but also their different crossing types are optimized during the search procedure.

The traffic assignment results may vary depending on where the new highway junction points (including the highway endpoints and intersection points) are connected on the existing road network. Such a case may be shown in the model application to the ICC project presented in Chapter 8. Recall, however, that the traffic assignment is not iteratively processed for every generated alignment over the successive generations if the assignment results with highways generated in the early generations (including the initial population) are not significantly varied (see section 8.2.3.1).

In order to find the preferable crossing types of the junction points, three highway crossing types (grade separation, interchange, and at-grade intersection) are pre-specified. Once a new alignment is generated and if it crosses any existing roads, construction costs as well as user travel costs with respect to the different crossing types are compared to find preferable ones. The best trade-off values among the various costs associated with those structures are found during the optimization process.

The grade-separation may be the most cost effective crossing type for construction; however, it does not allow any turning movements directly to the cross-roads. If light cross-traffic is expected, the grade-separation structure may be the most cost-effective one. However, the overall users travel costs of the network may increase if heavy cross-traffic is expected among the total traffic operating on the new highway. In such a case, a large fraction of the new highway traffic may experience longer travel because turning is structurally prohibited. On the other hand, the interchange may be the most expensive crossing type for construction, requiring a relatively large area and much added infrastructure. However, since it allows smooth cross-traffic without any interruption, the overall network travel cost may be reduced compared to the other crossing types. The at-grade intersection may cost less to construct than the interchange, while also providing turning movements from or to the new highways. However, if severe traffic delay is expected on the at-grade intersection due to heavy traffic (i.e., increase on turning movements), construction of other crossing types may be more cost-effective. Such a trade-off analysis is performed during the optimization process.

5. Generating Realistic Highway Alignments

In the previous HAO model (which is the predecessor of the proposed optimization model), only tangents and circular curves are used for generating horizontal alignments of a new highway. However, for high-speed highway alignments (or for rail alignments application), incorporation of spiral transitions to the horizontal curved section are strongly recommended in order to mitigate a sudden change in degree of curvature. By incorporating the transition spirals in horizontal curved sections, the resulting highway alignments from the model now become more realistic.

Besides the transition spirals, 3-leg structures (trumpet interchange, 3leg at-grade intersection and roundabout) for the highway endpoints, which are most commonly used in real highway projects, are modeled in this dissertation. Such work also helps the model produce more realistic highway alignments during the optimization procedure.

6. Sensitivity Analysis for Various Factors Associated with Highway Construction

Throughout the model application to real highway projects (the Brookeville and ICC case studies), it has been shown that the model can efficiently generate and evaluate numerous possible alignments, which reflect various user preferences and design standards, and even provide practical information of the resulting alignments to highway engineers and planners as a model output. In the case studies, several optimized alignments are found through the sensitivity analyses to various input parameters as well as the model objective function. The results indicate that many trade-off opportunities exist depending on the flexibility desired with the input parameters, and all alignment-sensitive costs associated with road construction should be considered and precisely formulated for a good highway optimization model. It is expected that such results can provide good insight in developing more comprehensive highway design models.

7. User-Friendly Interface

A number of data sets are required to process the proposed optimization model. For instance, the model users should specify alignment design standards, objectives, and preferences; in addition, spatial data sources (e.g., GIS maps) and traffic information (e.g., O/D trip matrices and a highway network) of the study area should be prepared. To greatly facilitate and speed up the selection of such input data sources needed for the model application, a user-friendly interface is also developed throughout this dissertation work. This interface helps the model users prepare the input GIS maps in machine-readable format and explore important considerations in the alignment selection process (such as environmentally-sensitive regions and topography of the study area). Furthermore, it displays characteristics of the resulting alignments in a more useful way. Detailed presentations of the alignment features (such as, horizontal and vertical profiles of the optimized solutions, their cost breakdowns by types, and environmental effects, and other performance measures) are provided in graphical and tabular forms. A detailed description of the user interface may be found in a MDSHA research report by Kang and Schonfeld (2007).

9.3 Recommendations for Future Extensions

Despite demonstrated capabilities of the proposed model, it can still benefit from various improvements in order to become more realistic and flexible in use. The following are some issues to be considered in the near future for enhancement of model performance.

1. Alignment Optimization for Varying Design Parameters

There is no limitation on the length of the highway alignments evaluated by the model as long as they can be generated and evaluated by the model within the specified search space. The design specifications of the resulting highways from the model are consistent along their alignments. It should be noted, however, that each road segment of a new highway alignment may not have the same design standard in reality. Due to terrain and land-use complexity and safety issue of surrounding environments, different design standards may be applied to different segments of the new highway. For instance, 60 mph design speed may apply for an alignment segment before it crosses an existing road; and only 50 mph afterward. Therefore, allowing different design standards for different segments of the solution alignments is recommended for improving the model's flexibility.

2. Preferable Number of PI's

The number of PI's is a key input parameter in the precision of the solution alignments since it affects location of horizontal and vertical curve sections as well as corresponding cost-components embedded in the model. In dense urban areas and areas with significant variation in the topography, a higher PI density will improve the possibilities for optimization, whereas in areas with slight variation in topography or land-use, fewer PI's will suffice. Therefore, PI density should be related to the complexity of the search space.

3. Distributed Computing

In a highway planning stage, several road construction projects may be considered together as a mater plan. In such a case, interdependency between the projects (such as budget issue, environmental issue, and traffic impact issue to the existing road system) may exist. Although a multi-stage optimization process, which sequentially optimizes alignments of each highway project, can be processed with the current model, optimizing the multiple highway projects simultaneously is desirable due to the interdependence. In order to optimize alignments of the multiple highways with the current model framework, alignments of each project should be simultaneously evaluated in addition to generating them. However, it is noted that a heavy computation burden may arise in such a case due to the time-consuming process required for alignment generation and evaluation.

Incorporation of a distributed computing technique (also known as the parallel computing that simultaneously uses multiple computing resources for solving a computational problem in a faster way) may be a good way for speeding up the optimization process. The idea is based on the fact that the process of solving a problem usually can be divided into smaller tasks, which may be carried out simultaneously with some coordination. Message-Passing Interface (MPI), which is one of the most common parallel computing techniques, may be incorporated into the alignment optimization model in the future for dealing with the multiple highway projects simultaneously.

4. Need for More Comprehensive Decision Making Approach

Some decision variables (e.g., political or environmental factors), which have a key role in the decision making of the highway selection process, may be too subjective or too intangible to quantify as monetary values. Thus, some additional decision making processes may be needed to represent those unquantifiable variables. The analytic hierarchy process (AHP) or a multi-objective decision making (MDM) analysis might be a logical decision making approach for the next model development phase.

5. Simulation-Based Approach

To estimate traffic impacts of the resulting highway alignments to the existing road system (i.e., travel time and vehicle operation costs of the highway users), analytic cost models are employed in the model. However, such analytic methods may oversimplify important details in the user cost estimation. A simulation-based approach (e.g., using CORSIM or PARAMIX) may be linked to the proposed model for more precise user cost estimation. However, it should be noted that because of a heavy computational burden expected, the simulation-based approach may not be necessary for all generated alternatives but only for several candidate alternatives (e.g., a set of the best ones found in every generation).

6. Optimization with Different Levels of GIS Inputs Prepared

While the model is capable of handling cases with complex topography and land-use, it may be difficult to obtain sufficient information required for such applications from real GIS databases directly. Extra time and cost may be needed for preparation of the detailed spatial and land-use information in a machine-readable format even though GIS's are widely used in the world. If the search for optimized alignments has to be performed in a complex geographic space (e.g., in urban highway networks), detailed GIS inputs may be necessary for obtaining convincing and accurate results from the model. With more detailed GIS data, more accurate optimized results can be obtained from the model. However, if preparation of the detailed GIS inputs is too expensive at the very beginning of a road planning stage, a model search with cheap GIS information may suffice. A rough idea of the optimized alignments can be obtained from the basic search. A model search with a full GIS evaluation process, which provides more precise optimization results, may be needed after all the GIS inputs are fully prepared.

Appendix A

Table A.1 TAZ IDs Aggregated to Hypothetical Centroids for ICC Case Study

Centroid ID	TAZ IDs Aggregated to Centroids	No. of TAZs
1	526~543, 557~576, 605, 607	29
2	512, 516, 517, 530~533, 537~539	10
3	471, 473, 480~483, 487~489, 555, 556, 580	12
4	544~554, 577~579	14
5	467~469, 474, 475, 518, 519, 522~525, 534~536	14
6	376, 387, 388, 471	4
7	407, 470, 476~479, 485	7
8	397, 404~418, 432, 491~493	19
9	381~385, 389~396, 399, 401, 472	15
10	196, 197, 320~328, 336~339, 377~382, 386, 1465~1471	28
11	198~200, 329~331, 340~349, 398, 402, 403, 419	19
12	206, 215~217, 221, 332~335, 347, 350, 351, 420~423, 433, 434	18
13	486, 490, 496, 497, 581, 588, 589	7
14	500, 505, 592	3
15	501~504	4
16	582, 622, 623, 1089	4
17	583~585, 591, 593	5
18	1085~1087	3
19	506~509, 1083, 1084, 1092, 1094, 1095, 1099	10
20	460~466, 781, 783~785, 865~867, 871	15
21	870, 873~875, 877, 878, 1081, 1082, 1091, 1093, 1096~1098	13
22	869, 872, 876, 879, 880, 888~891, 1080, 1090	11
23	884~887, 892~898	11
24	1118, 1119, 1122~1126, 1131, 1133~1136, 1140, 1141	14
25	789~793, 881~883	8
26	670~677	8
27	678~685, 787, 788, 794~815	32
28	218~224, 352~358, 424~431, 440, 477, 478	24
29	235~241, 359~367	16
30	243, 244, 368~374, 451, 454, 455, 642, 648, 650, 655, 656, 659	17
31	375, 456~459, 640, 641, 782	8
32	643~645, 647, 657	5
33	435~446, 452, 453, 494, 498, 499	16

Table A.2 AM-Peak O/D Trips between the Aggregated Centroids for ICC Case Study (trips/hr)

O/D	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	Sum
1	0	1497	639	2751	1095	81	375	756	546	219	387	120	54	9	12	147	75	6	21	39	15	9	3	9	6	6	15	108	108	45	39	6	60	9258
2	1404	0	342	852	885	48	204	384	276	111	198	60	27	3	6	24	18	3	9	18	6	6	3	6	3	3	9	60	54	27	21	3	33	5106
3	636	345	0	1029	513	69	270	450	255	87	159	54	75	9	12	60	51	3	15	21	12	6	3	6	3	3	6	60	45	21	18	3	72	4371
4	2577	798	876	0	822	60	309	534	399	162	255	84	93	15	21	246	138	9	30	33	21	9	3	9	6	3	12	81	75	33	27	6	78	7824
5	765	816	402	642	0	90	378	645	399	156	264	84	42	6	9	24	21	3	12	24	9	6	3	6	6	3	12	84	72	39	30	6	60	5118
6	114	69	84	90	126	0	81	237	294	126	138	36	6	3	3	3	3	3	6	9	3	3	3	3	3	3	6	33	30	15	12	3	15	1563
7	252	147	186	219	336	33	0	720	231	75	138	60	57	9	12	12	21	3	15	21	6	3	3	3	3	3	6	72	36	21	15	3	84	2805
8	474	249	309	369	498	87	738	0	1032	264	735	375	258	24	54	21	63	6	66	138	27	18	3	18	12	6	27	552	192	111	93	12	792	7623
9	429	225	198	342	372	180	297	1686	0	513	981	267	39	6	12	12	12	3	30	69	18	15	3	18	12	6	27	258	192	108	78	9	123	6540
10	183	102	75	147	165	129	123	525	708	0	1605	318	12	3	6	3	6	3	18	42	18	15	3	21	9	6	30	168	195	72	48	9	42	4809
11	222	123	96	171	189	60	144	1134	876	1113	0	852	39	6	18	9	12	3	45	111	30	24	6	27	18	9	42	540	432	162	120	15	150	6798
12	90	48	45	72	84	18	84	603	294	315	1437	0	39	9	18	6	12	3	33	87	33	27	3	24	15	9	33	777	459	138	93	15	270	5193
13	102	51	138	153	114	9	186	600	84	27	108	75	0	39	66	21	78	6	60	87	33	12	3	12	6	3	12	192	57	36	42	6	636	3054
14	15	6	15	24	15	3	21	57	9	6	18	15	39	0	60	9	48	9	63	36	27	12	3	12	3	3	3	33	15	12	18	3	63	675
15	21	12	24	33	27	3	42	153	30	15	63	39	72	54	0	9	36	6	222	192	57	27	3	21	6	3	12	123	57	57	84	6	219	1728
16	333	66	126	483	90	3	39	72	39	15	33	12	24	12	9	0	90	81	123	15	81	21	3	21	3	3	3	18	12	6	6	3	27	1872
17	171	48	120	297	87	3	75	195	39	12	48	30	108	69	51	108	0	18	81	57	54	21	3	30	3	3	6	69	30	18	27	3	123	2007
18	18	3	9	21	6	3	6	15	6	3	6	3	9	9	6	51	21	0	894	21	636	162	3	120	6	3	9	9	9	6	6	6	12	2097
19	36	15	24	42	30	6	33	132	57	30	111	51	45	39	120	72	48	798	0	597	3198	702	21	492	33	12	63	114	105	102	162	24	126	7440
20	36	18	21	30	36	6	39	240	99	51	189	105	54	21	99	6	24	12	495	0	516	303	66	105	252	45	300	261	189	312	480	93	285	4788
21	215	209	12	221	15	3	15	54	33	1021	63	30	18	18	27	33	27	375	2763	372	0	2157	102	1053	108	27	1204	57	63	78	78	54	39	10544
22	209	206	6	209	9	3	9	30	24	15	42	24	6	6	12	9	9	90	495	255	1947	0	267	570	153	36	312	39	42	60	57	57	21	5229
23	203	203	3	203	6	3	3	12	9	9	21	9	3	3	3	3	3	6	27	129	294	570	0	201	171	27	333	15	18	30	24	36	6	2586
24	209	206	6	209	12	3	9	39	30	21	57	27	6	6	12	6	12	78	450	135	1599	1011	120	0	108	36	372	48	57	54	48	48	15	5049
25	6	3	3	3	3	3	3	12	9	6	15	9	3	3	3	3	3	3	18	156	99	108	57	39	0	24	231	18	15	42	30	36	6	972
26	3	3	3	3	3	3	3	9	6	6	12	9	3	3	3	3	3	3	6	39	27	27	9	15	39	0	165	12	21	87	24	69	3	624
27	18	9	9	15	21	3	15	69	54	48	96	51	6	3	9	3	3	6	54	459	339	393	183	288	594	228	0	93	96	222	153	258	27	3825
28	99	54	57	78	96	21	99	687	315	171	723	660	81	15	45	6	24	3	75	210	51	39	6	36	30	15	63	0	861	336	216	24	504	5700
29	63	33	30	48	57	15	39	237	168	150	495	327	18	6	18	3	9	3	54	135	54	39	6	42	27	24	63	597	0	444	168	33	87	3492
30	51	30	27	42	60	12	42	267	186	102	312	192	24	9	36	3	9	3	99	435	132	99	18	48	111	114	246	528	678	0	579	231	129	4854
31	36	21	18	27	36	9	27	153	105	54	180	105	21	9	39	3	12	3	120	510	117	87	15	42	81	36	159	255	216	522	0	114	111	3243
32	3	3	3	3	6	3	3	12	9	9	18	12	3	3	3	3	3	3	15	93	54	45	12	18	60	66	180	21	33	174	81	0	12	966
33	87	48	111	105	123	15	207	1410	210	72	330	366	549	51	153	18	75	6	141	330	57	27	6	24	15	6	33	963	183	141	159	18	0	6039
Sum	9080	5666	4017	8933	5937	987	3918	12129	6831	4984	9237	4461	1833	480	957	939	969	1560	6555	4875	9570	6003	945	3339	1905	774	3994	6258	4647	3531	3036	1212	4230	143792

Source: 2010 O/D trip tables from MWCOG

Table A.3 PM-Peak O/D Trips between the Aggregated Centroids for ICC Case Study (trips/hr)

O/D	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	Sum
1	0	1496	638	2749	1093	79	374	754	544	219	385	119	54	9	10	145	73	5	21	37	15	209	202	208	6	4	15	108	108	45	38	6	59	9827
2	1404	0	341	852	885	46	203	383	274	110	197	60	26	2	5	24	17	1	8	18	5	204	201	204	3	2	8	58	54	25	20	3	32	5675
3	634	343	0	1027	513	69	269	449	255	85	159	53	74	8	12	59	49	2	15	19	10	4	1	6	3	2	6	58	43	21	16	2	70	4336
4	2575	797	876	0	821	60	308	533	398	160	253	83	93	15	19	245	138	8	28	31	19	209	201	209	5	3	10	79	75	31	27	5	76	8390
5	764	816	402	641	0	89	378	645	397	154	263	83	41	5	8	22	19	1	12	24	7	5	1	6	4	2	10	82	71	39	28	4	58	5081
6	112	69	84	88	126	0	80	236	293	126	137	34	6	1	1	3	2	1	4	9	3	2	1	3	2	1	4	32	30	15	11	2	15	1533
7	252	147	184	218	336	33	0	720	230	74	138	60	56	7	12	10	19	1	13	20	6	3	1	3	2	1	5	71	36	21	15	2	82	2778
8	472	247	308	367	498	86	738	0	1030	263	734	373	257	22	54	21	62	4	66	138	25	16	3	18	11	5	27	552	192	111	91	10	792	7593
9	429	225	196	341	370	178	297	1685	0	512	979	265	37	4	10	11	11	1	29	67	17	13	3	18	10	5	26	257	192	107	76	9	122	6502
10	182	102	74	146	164	127	121	525	708	0	1605	318	12	2	4	3	4	1	17	42	1018	13	2	19	9	6	30	167	195	71	48	9	40	5784
11	220	121	96	171	189	60	144	1134	874	1112	0	852	37	5	17	7	12	1	45	110	30	23	5	27	17	8	42	539	430	162	120	14	149	6773
12	90	48	44	71	84	17	84	602	294	314	1437	0	39	7	16	4	10	1	33	86	32	25	3	23	15	8	31	775	457	136	93	13	269	5161
13	101	49	137	153	114	8	185	600	82	26	108	74	0	39	64	19	77	5	59	85	31	12	2	12	4	2	10	191	56	34	40	5	636	3020
14	15	5	14	23	14	1	19	55	9	4	17	13	39	0	59	8	48	7	62	36	27	12	1	12	2	1	3	32	14	12	17	2	63	646
15	21	10	24	33	25	2	40	153	28	13	62	37	70	53	0	7	35	4	220	190	56	25	3	21	6	3	12	122	56	57	83	5	218	1694
16	332	64	126	482	89	3	38	71	37	14	31	11	24	10	8	0	89	79	123	13	79	21	1	20	1	1	3	16	11	5	6	1	26	1835
17	171	47	119	295	86	3	73	193	37	12	46	30	107	68	49	106	0	18	79	56	53	19	1	29	3	1	6	69	29	18	27	3	122	1975
18	16	3	7	19	6	1	6	15	4	2	6	3	7	9	6	51	20	0	892	21	635	160	3	120	4	2	9	8	7	5	6	4	12	2069
19	34	13	23	40	28	4	33	132	57	28	109	51	43	38	119	72	48	798	0	596	3198	701	19	490	33	10	61	114	105	100	162	23	124	7406
20	36	18	21	30	36	6	39	238	97	50	188	103	52	21	97	5	23	10	494	0	516	302	65	105	251	43	299	259	189	310	479	91	285	4758
21	15	7	11	19	13	3	14	53	31	19	61	29	16	16	27	33	25	373	2761	372	0	2157	101	1051	108	27	203	57	61	77	77	54	39	7910
22	8	5	5	9	9	2	7	30	22	15	40	22	6	6	12	8	8	89	495	254	1947	0	267	568	153	35	311	38	41	60	56	56	19	4603
23	3	2	2	2	4	1	3	12	9	7	19	9	2	1	2	1	1	4	27	128	294	568	0	200	170	25	331	15	16	28	22	34	5	1947
24	9	5	5	8	12	2	8	38	30	21	57	27	6	6	11	6	10	78	449	133	1598	1010	120	0	107	36	371	46	56	53	47	47	14	4426
25	4	2	2	3	3	1	2	11	9	5	14	7	2	1	2	1	1	2	16	155	98	106	56	37	0	24	230	16	14	40	30	34	5	933
26	2	1	1	2	2	1	2	7	5	5	11	7	1	1	1	1	1	1	5	39	26	26	9	14	38	0	164	11	21	87	22	68	3	585
27	17	9	7	13	21	3	13	68	52	48	96	50	5	2	7	1	3	5	54	457	1337	391	183	288	593	227	0	92	95	220	153	256	26	4792
28	97	54	56	78	94	20	99	686	315	169	722	659	79	14	45	6	23	2	75	209	51	38	6	35	30	14	63	0	860	334	215	23	503	5674
29	61	33	28	47	55	13	37	236	167	148	493	326	18	5	17	2	8	2	52	135	52	37	6	40	27	23	62	597	0	444	167	33	86	3457
30	51	30	25	40	58	12	41	266	184	100	311	190	22	8	34	3	9	2	99	434	132	97	18	48	109	114	245	527	677	0	579	231	129	4825
31	35	19	16	27	36	7	25	151	103	52	180	103	21	8	38	3	11	3	118	510	115	85	14	40	81	34	158	253	215	522	0	113	110	3206
32	3	2	2	3	4	1	3	12	9	7	18	11	2	1	2	1	1	1	13	91	52	43	12	18	59	64	178	20	31	174	81	0	10	929
33	85	46	110	105	123	13	206	1409	209	72	330	365	548	51	153	16	75	6	141	330	56	27	4	24	13	6	31	962	183	140	159	16	0	6014
Sum	8250	4835	3984	8102	5911	951	3889	12102	6793	3946	9206	4427	1802	445	921	904	932	1516	6525	4845	11540	6563	1515	3916	1879	739	2964	6223	4620	3504	3011	1178	4199	142137

Source: 2010 O/D trip tables from MWCOG

Table A.2 OFF-Peak O/D Trips between the Aggregated Centroids for ICC Case Study (trips/hr)

O/D	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	Sum
1	0	1496	638	2749	1093	79	374	754	544	219	385	119	54	9	10	145	73	5	21	37	15	9	2	8	6	4	15	108	108	45	38	6	59	9227
2	1404	0	341	852	885	46	203	383	274	110	197	60	26	2	5	24	17	1	8	18	5	4	1	4	3	2	8	58	54	25	20	3	32	5075
3	634	343	0	1027	513	69	269	449	255	85	159	53	74	8	12	59	49	2	15	19	10	4	1	6	3	2	6	58	43	21	16	2	70	4336
4	2575	797	876	0	821	60	308	533	398	160	253	83	93	15	19	245	138	8	28	31	19	9	1	9	5	3	10	79	75	31	27	5	76	7790
5	764	816	402	641	0	89	378	645	397	154	263	83	41	5	8	22	19	1	12	24	7	5	1	6	4	2	10	82	71	39	28	4	58	5081
6	112	69	84	88	126	0	80	236	293	126	137	34	6	1	1	3	2	1	4	9	3	2	1	3	2	1	4	32	30	15	11	2	15	1533
7	252	147	184	218	336	33	0	720	230	74	138	60	56	7	12	10	19	1	13	20	6	3	1	3	2	1	5	71	36	21	15	2	82	2778
8	472	247	308	367	498	86	738	0	1030	263	734	373	257	22	54	21	62	4	66	138	25	16	3	18	11	5	27	552	192	111	91	10	792	7593
9	429	225	196	341	370	178	297	1685	0	512	979	265	37	4	10	11	11	1	29	67	17	13	3	18	10	5	26	257	192	107	76	9	122	6502
10	182	102	74	146	164	127	121	525	708	0	1605	318	12	2	4	3	4	1	17	42	1018	13	2	19	9	6	30	167	195	71	48	9	40	5784
11	220	121	96	171	189	60	144	1134	874	1112	0	852	37	5	17	7	12	1	45	110	30	23	5	27	17	8	42	539	430	162	120	14	149	6773
12	90	48	44	71	84	17	84	602	294	314	1437	0	39	7	16	4	10	1	33	86	32	25	3	23	15	8	31	775	457	136	93	13	269	5161
13	101	49	137	153	114	8	185	600	82	26	108	74	0	39	64	19	77	5	59	85	31	12	2	12	4	2	10	191	56	34	40	5	636	3020
14	15	5	14	23	14	1	19	55	9	4	17	13	39	0	59	8	48	7	62	36	27	12	1	12	2	1	3	32	14	12	17	2	63	646
15	21	10	24	33	25	2	40	153	28	13	62	37	70	53	0	7	35	4	220	190	56	25	3	21	6	3	12	122	56	57	83	5	218	1694
16	332	64	126	482	89	3	38	71	37	14	31	11	24	10	8	0	89	79	123	13	79	21	1	20	1	1	3	16	11	5	6	1	26	1835
17	171	47	119	295	86	3	73	193	37	12	46	30	107	68	49	106	0	18	79	56	53	19	1	29	3	1	6	69	29	18	27	3	122	1975
18	16	3	7	19	6	1	6	15	4	2	6	3	7	9	6	51	20	0	892	21	635	160	3	120	4	2	9	8	7	5	6	4	12	2069
19	34	13	23	40	28	4	33	132	57	28	109	51	43	38	119	72	48	798	0	596	3198	701	19	490	33	10	61	114	105	100	162	23	124	7406
20	36	18	21	30	36	6	39	238	97	50	188	103	52	21	97	5	23	10	494	0	516	302	65	105	251	43	299	259	189	310	479	91	285	4758
21	15	7	11	19	13	3	14	53	31	19	61	29	16	16	27	33	25	373	2761	372	0	2157	101	1051	108	27	203	57	61	77	77	54	39	7910
22	8	5	5	9	9	2	7	30	22	15	40	22	6	6	12	8	8	89	495	254	1947	0	267	568	153	35	311	38	41	60	56	56	19	4603
23	3	2	2	2	4	1	3	12	9	7	19	9	2	1	2	1	1	4	27	128	294	568	0	200	170	25	331	15	16	28	22	34	5	1947
24	9	5	5	8	12	2	8	38	30	21	57	27	6	6	11	6	10	78	449	133	1598	1010	120	0	107	36	371	46	56	53	47	47	14	4426
25	4	2	2	3	3	1	2	11	9	5	14	7	2	1	2	1	1	2	16	155	98	106	56	37	0	24	230	16	14	40	30	34	5	933
26	2	1	1	2	2	1	2	7	5	5	11	7	1	1	1	1	1	1	5	39	26	26	9	14	38	0	164	11	21	87	22	68	3	585
27	17	9	7	13	21	3	13	68	52	48	96	50	5	2	7	1	3	5	54	457	1337	391	183	288	593	227	0	92	95	220	153	256	26	4792
28	97	54	56	78	94	20	99	686	315	169	722	659	79	14	45	6	23	2	75	209	51	38	6	35	30	14	63	0	860	334	215	23	503	5674
29	61	33	28	47	55	13	37	236	167	148	493	326	18	5	17	2	8	2	52	135	52	37	6	40	27	23	62	597	0	444	167	33	86	3457
30	51	30	25	40	58	12	41	266	184	100	311	190	22	8	34	3	9	2	99	434	132	97	18	48	109	114	245	527	677	0	579	231	129	4825
31	35	19	16	27	36	7	25	151	103	52	180	103	21	8	38	3	11	3	118	510	115	85	14	40	81	34	158	253	215	522	0	113	110	3206
32	3	2	2	3	4	1	3	12	9	7	18	11	2	1	2	1	1	1	13	91	52	43	12	18	59	64	178	20	31	174	81	0	10	929
33	85	46	110	105	123	13	206	1409	209	72	330	365	548	51	153	16	75	6	141	330	56	27	4	24	13	6	31	962	183	140	159	16	0	6014
Sum	8250	4835	3984	8102	5911	951	3889	12102	6793	3946	9206	4427	1802	445	921	904	932	1516	6525	4845	11540	5963	915	3316	1879	739	2964	6223	4620	3504	3011	1178	4199	140337

Source: 2010 O/D trip tables from MWCOG

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