Developing a Tool for the Design and Cost Optimization of Permeable Pavements in the Planning Stage of Stormwater Management

by

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ABSTRACT

Stormwater runoff, when not managed properly, can represent a threat to the environment especially in today's urban setting. For small pavement applications such as parking lots, conventional pavements and best management practices such as detention ponds are commonly used to address this issue. However, for such applications, there is currently an increase of interest in more sustainable options like Permeable Pavement (PP), which is a Green Infrastructure (GI) practice that helps store and treat stormwater runoff as well as support traffic loadings. Consequently, designers look for more efficient and economical ways to include PPs and other GI practices in their planning for stormwater management and compete with conventional systems, which are still perceived as the least expensive options.

The product of this research is a tool called the "Cost Optimization Tool for Permeable Pavements (COTPP)" that will benefit Alabama municipalities and GI design agencies because it helps optimize the cost of PP systems for stormwater management. This tool contains the most reliable design methods (hydrological and/or structural) required to design three types of PP, which are Pervious Concrete (PC), Porous Asphalt (PA), and Permeable Interlocking Concrete Pavers (PICP). The COTPP also contains a cost optimization algorithm that helps combine permeable pavements with other types of GI, and conventional pavements to identify which combination will minimize costs and compete with conventional systems. Although the COTPP only supports three GI techniques, the algorithm was designed to be expanded to include other techniques in the future. Further, the COTPP was developed in a Microsoft Excel format because it is a computing tool that is available to most design engineers and decision makers. This will help reduce the need to invest in and learn complex modeling or optimization packages.

A sensitivity analysis of the COTPP was performed to check the reliability of the design results and the efficiency of the tool. The analysis consisted of designing the three types of permeable pavements according to varying design inputs and comparing the results to recommendations from design guidelines or results from existing tools. The existing tools include PerviousPave from the American Concrete Pavement Association (ACPA) and the guidelines are from the National Asphalt Pavement Association (NAPA) and the Interlocking Concrete Pavement Institute (ICPI). For the structural design, it was found that the average percent differences between the design results of the COTPP and the results from the existing tools or guidelines are 4.4%, 7.2%, and 4.8% for PC, PA, and PICP, respectively. For the hydrological design, the results were the same for PC, PA, and PICP because they have the same design method in common. The average percent difference obtained between the design results of the COTPP and the results from existing tools is 15.1%. This number was particularly high because the hydrological design method used in the COTPP applies a factor of safety and a few different inputs which lead to more conservative results. It was concluded that the COTPP is a reliable tool that can be used confidently for the design of PPs.

A case study was conducted to test the usefulness of the tool in terms of cost optimization of PPs. In the case study, an existing parking lot located in Alabama was selected and its construction costs were obtained from bid documents. The existing parking lot site was composed of PICP, a bioretention, and a HMA pavement. The costs were used in the COTPP to evaluate the capability of the tool to reduce the final construction cost of $305,918. The results showed that the actual cost of the parking lot ($305,918) could be reduced to as low as $238,790 using an alternative design where PICP occupies 11% of the parking lot area instead of the original 21% and the bioretention treats 90% of the total treatment volume. It was concluded, based on the results, that the cost of permeable pavements could effectively be optimized by the COTPP developed for Alabama designers and engineers during their planning stage. The unit costs of individual components (aggregate base, PC, PA etc.) that compose the parking lot and design constraints are the controlling factors in the optimization process.

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CHAPTER 1: INTRODUCTION

1.1 Background

Today, urbanization is considered one of the main factors that contribute to the increase in runoff volume and peak flow, the decrease of groundwater water recharging rate, and the accumulation of pollutant loads carried by the runoff. This is an issue because surface waters receive sediment, nutrients and heavy metals that are carried by urban runoff. As a consequence, surface waters become polluted which can cause the quality of groundwater to degrade. Stormwater runoff was part of the top three sources of pollution in lakes, ponds, reservoirs, and estuaries in 2000 (Bean et al., 2007). Many state agencies developed stormwater management programs to help reduce stormwater runoff associated with transportation facilities. As a result, designers are trying to find ways to use PPs in the way that could replace more traditional stormwater best management practices (BMPs). PPs contain voids that allow storm water runoff to pass through and get stored temporarily in the underlying reservoir layer before it infiltrates the native soil. There are different categories of PPs, but the three main types are Pervious Concrete (PC), Porous Asphalt (PA), and Permeable Interlocking Concrete Pavers (PICP) as shown in Figure 1. They all have different surface materials, but have a similar structure composed of a surface pavement layer, an aggregate reservoir layer, a filter fabric, and the native soil as shown in Figure 2 (City of Birmingham, 2019).

(a) (b) (c)

Figure 1. Interlocking concrete pavers (a), pervious concrete (b), and porous asphalt (c) (Van Dam et al. 2015)

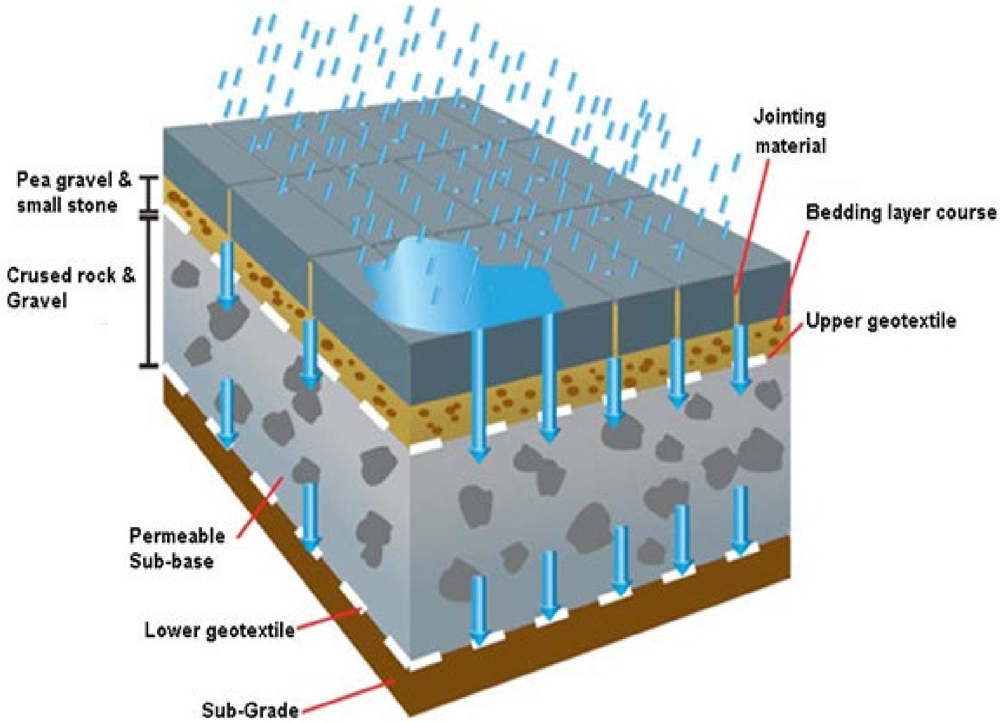

Figure 2. Structure of Permeable Pavements (Tota-Maharaj & Scholz, 2010)

PPs have multiple advantages such as a reduction in runoff volume, increase of aesthetic value, better quality of water treatment, and the capability to remove 80% of total suspended solids (TSS). However, PPs have some limitations such as high-cost maintenance, restriction to low traffic areas because of low structural capacity, difficulties for handicap access, potential soils with low infiltration capacity, and low effectivity on steep slopes. To determine the thickness of the reservoir layer, both a structural and a hydrological design are required. The stone aggregate reservoir layer helps retain storm water and support structurally the traffic loads applied on the pavement. If the underlying soil has a low infiltration rate, an underdrain should be constructed in the reservoir layer to collect some or all of the runoff. This is called a "Level 1 design". In this study, the focus is on the design of PPs without underdrain (Level 2 design), which means that the underlying soil needs to have a moderate or high permeability. In addition to the ability to store and treat water runoff that falls on the surface, PPs can also receive runoff from small adjacent impervious areas such as impermeable driving lanes or rooftops (City of Birmingham, 2019). To perform effectively, PPs need to satisfy some important criteria such as the ability to handle traffic loads and speed, the ability to infiltrate and store stormwater, and the satisfactory permeability of the subgrade soil (Weiss et al., 2017). PPs can be more economical than conventional pavements because they reduce the need for stormwater ponds, curb and gutter, and catch basins (Dylewski et al., n.d.).

PPs can potentially be great assets to increase the sustainability of stormwater management in Alabama because they take into considerations the quality of the environment through the treatment and reduction of surface runoff from urban development (environmental and social benefits), and they can be more economical than conventional pavements in a long term (economic benefits). To be widely implemented, PPs must be able to compete with conventional practices in terms of both performance and cost. PPs are still perceived as expensive options. For instance, 37 designers surveyed by Harvey J. et al. (2017) stated that high cost is the second top issue after maintenance that affect the implementation of PPs. However, there are multiple ways that PP systems can be designed to reduce cost while still meeting their performance requirements, especially considering the wide range of available GI practices and design options. There are some existing tools that help determine the costs of PPs, but none of them help optimize their costs through alternative designs such as the combination of PPs with other GIs or any other methods. Therefore, there is a need for a practical optimization tool to help design and minimizing costs of PPs, which will contribute to their promotion and maximize the environmental benefits received from investing in them.

1.2 Research Objectives

The purpose of this research study was to develop a practical tool that will benefit Alabama municipalities and owner agencies by helping achieve cost optimization for PP systems for stormwater management. The objectives assigned to this research are:

- To create a user-friendly tool that is accessible and support the design of the three main types of PP, which are PC, PA, and PICP. It is important to note that no agency possesses a tool that help design these three types of PPs at once.
- To develop an optimization algorithm that assist in identifying which, or in what combinations, PP systems can be used with conventional pavements, bioretention, and infiltration trench to minimize costs.
- To ensure that the writing of the algorithm in the tool is understood by its users and can be expanded to include other techniques in the future.

1.3 Scope

To achieve the goals set for this research, several steps were taken through literature and computer modelling resources. The tool obtained from this study is called "the Cost Optimization Tool for Permeable Pavements (COTPP)" and includes the hydrological and structural design methods of each type of PP. Only three categories are considered in the tool: PC, PA, and PICP. Many design methods exist, but only the most comprehensive methods according to Weiss et al. (2017) were selected and inserted in the tool. The design of underdrains to transfer stormwater runoff from the PP to another storage is not considered within the scope of this study. In this tool, the cost optimization model created relies on the combination of PPs with conventional non-pervious pavements and two GI practices which are bioretention and infiltration trench. This tool allows the user to choose the most appropriate and economical design option for a specific project.

The tool's intended purpose is to help stormwater managers in Alabama with project planning. To promote the use of PPs in stormwater management projects, COTPP was developed for use in MS Excel, a common computing tool that most, if not all, design engineers and decision makers have available. Thus, there will be no need to invest in and learn complex modeling or optimization packages. In addition, the tool was designed to be expanded and include other stormwater BMP's and the capability to compare GI with conventional infrastructure. The conclusions of this study are considered applicable for small pavement project applications such as: car parking areas and access lanes; shopping center entrance and service lanes; interior lanes;; entrance and exterior lanes; collectors and shoulders for major or minor arterials, etc.

1.4 Organization of Thesis

This thesis was organized in six (6) chapters described as follows: Chapter 1 is the "Introduction" that gives a background on the reasons that led to this research as well as a description of the importance of using PPs for stormwater management. It also describes the objectives and the scope of work of this research. Chapter 2 is a "Literature Review" that provides a summary of previous studies and existing findings from other researchers concerning PP design, performance, challenges, costs, design tools, and cost optimization tools. Chapter 3 presents the "Methodology" that gives an overview of the tool, outlines all the steps and processes followed to create the tool. Chapter 4 discusses the sets of procedures used to perform a "Sensitivity Analysis" and a "Case Study". Chapter 5 provides the "Presentation of Results and Discussion" of the outcomes of the case study and sensitivity analysis. Chapter 6 summarizes all the "Conclusions" of this study and "Recommendations" for future research work. All these chapters are followed by Appendices A through F that show all additional information necessary to understand this study.

CHAPTER 2: LITERATURE REVIEW

2.1 Green Infrastructure

The concepts of green infrastructure were introduced in the mid-1980s proposals for best management practices in order to achieve better water quality and reduce runoff volume (Schueler, 1987). Cities across the country continued to have a lot of stormwater management problems and significant capital investment is required to enhance the performance of existing conventional infrastructures or to build new ones. Therefore, there is a need for a re-evaluation of the investment strategies and the environmental programs as cities try to become more sustainable. The strategies used for improvement of stormwater infrastructure so far have focused essentially on capacity and conveyance while neglecting investments (Kloss, 2008). To minimize costs, green infrastructure started to become the new alternative that many cities chose to manage stormwater, enhance their infrastructure, and provide multiple environmental benefits. Those advantages allow municipalities to have a framework for sustainable infrastructure management and use their limited economic resources more efficiently (Kloss, 2008).

Green infrastructure mimics natural processes via the use of trees and vegetation, or engineered systems, which help manage stormwater in a more comprehensive way. Stormwater introduced to a green infrastructure system usually leaves by various paths such as infiltration, evapotranspiration, or retention by capturing and treating stormwater runoff as it infiltrates the soil. Green infrastructure provides additional environmental benefits such as energy savings, improved air quality, aesthetic improvements, reduced urban temperatures, and a potential strategy for reducing carbon footprints (Kloss, 2008). One type of green infrastructure is permeable pavement.

2.2 Permeable Pavement Types

PP is a type of GI that is designed to allow the storm water runoff, usually the "first flush", to be stored temporarily in its reservoir before it infiltrates the native soil. The first flush is the first load (usually high in pollutants) of stormwater during a rain event. The three main types of PPs (PC, PA, PICP) are considered in this project. The various characteristics of these PPs and their performance history are discussed below.

2.2.1 Pervious Concrete

Pervious Concrete (PC), according to Dylewski et al., n.d., is a composition of Portland cement, fly ash, coarse washed aggregate and water and has a void content that ranges between 15 and 25%. This void content is larger than the void content (5%) of conventional pavements. PC can reach a compressive strength of 3000 psi after seven days of curing and an infiltration rate of 300 in./hr. PC is well known for its ability to maintain its structural strength even during extremely hot weather, therefore it is widely used in hot climate locations like the state of Alabama (Dylewski et al., n.d.). The thickness of the PC layer can get as high as 8 in. depending on the design. Pervious concrete is usually constructed over a 1 to 2 in. thick choking stone layer composed of 0.5 in. diameter fine aggregate that contribute to the stabilization of the PC and the infiltration of water into the reservoir layer. For the base/subbase layer or reservoir layer, No. 2 or No. 3 stone aggregate are recommended with a minimum reservoir depth of 9 in. (Dylewski et al., n.d.). According to City of Birmingham (2019), the reservoir layer or subbase layer shall be composed of clean, washed No. 57 stone aggregate, even though No. 2 stone is usually preferred because of its structural stability and the extra storage space it can provide. A geotextile filter fabric or sand layer of 6 in. are recommended to help stabilize the base/subbase layer and minimize the compaction of the underlying native soil also called the subgrade (Dylewski et al. n. d.). Figure 3 shows a sketch of a typical PC cross section.

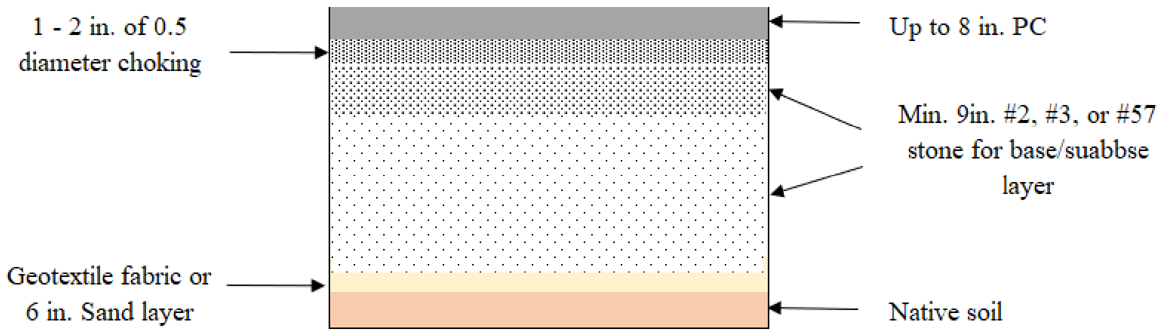

Figure 3. Typical PC cross section

2.2.2 Porous Asphalt

Porous Asphalt (PA), according to Dylewski et al. (n.d.), is a mixture of fine/coarse sand and bituminous-based binder. It has a void content of approximately 15 – 20% to ensure proper water infiltration and is appropriate for cold weather locations because of its ability to maintain void space when it freezes. This helps stormwater runoff continue infiltrating into the reservoir layer and reduce freezing and thawing, which also decreases frost heaving action experienced in conventional asphalt pavements. PA has a lifespan of approximately 30 years but has a lower modulus and higher costs compared to conventional asphalt pavements. It is the least expensive among the three types of PP and can get as thick as 7 in. even though 2 to 4 in. is a typical range (Dylewski et al., n.d.). It is also usually built over a 1 – 2 in. thick choking stone layer composed of small open-graded aggregate (Dylewski et al., n.d.; EPA, 1980). The No. 57 stone and No. 2 stone aggregate are recommended respectively for the first 3 to 4 in. and the remaining of the base/subbase layer. Like PC, a geotextile filter fabric or sand layer are recommended between the reservoir layer and the subgrade (Dylewski et al., n.d.). A sketch of a typical PA cross section is shown in Figure 4.

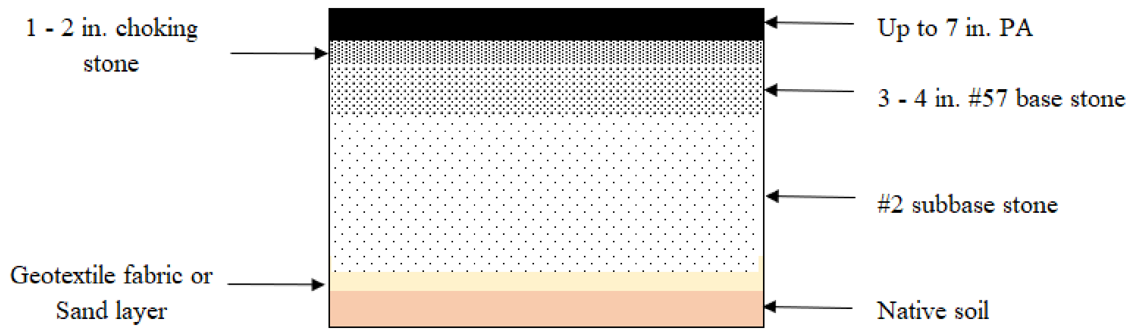

Figure 4. Typical PA cross section

2.2.3 Permeable Interlocking Concrete Pavers (PICP)

Permeable Interlocking Concrete Pavers (PICP), according to Dylewski et al., n.d., are concrete blocks with fine aggregate between them and a void content of 8 – 20%. They have the highest structural strength compared to other PPs and are roughly 3 in. thick. The openings between the pavers are always filled with No. 8 or No. 89 stone aggregate. PICP can be very aesthetically pleasing because of their varieties of shapes, sizes, and colors available. PICP are usually ready for traffic immediately after installation because they are installed mechanically and do not require time to cure. PICP are usually installed over a 1.5 to 2 in. thick chocking stone layer composed of No. 8 or No. 89 stone, 4 in. thick of No.57 stone representing the base layer, a subbase layer composed of No. 2 stone, a geotextile filter fabric or sand layer, and the underlying native soil (Dylewski et al., n.d.). Figure 5 shows a sketch of a typical PICP cross section.

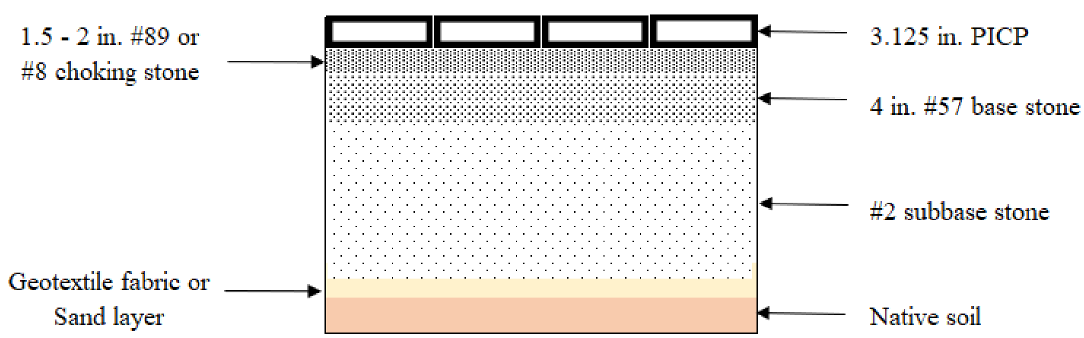

Figure 5. Typical PICP cross section

2.3 Performance History

To understand how existing PP systems perform structurally, numerous studies were performed. More than 200 PA pavements were designed and constructed by companies since the 1980s. For the PA pavements that were built following construction practices, no pavement failure was reported. It was found that PP is one of the most effective stormwater management options in terms of water treatment (NAPA, 2008). AlShareedah & Nassiri (2019) used a lightweight deflectometer to test 14 PC pavements that have thicknesses ranging from 4 to 12 in. (102 to 305 mm). The results of the back-calculation showed that the elastic moduli or modulus of elasticity of the PC pavements range from 1.74×10^6 to 3.05×10^6 psi (12 to 21 GPa). Vancura et al. (2011b) conducted a study to identify subsurface distresses on 29 PC pavements in a hard freeze environment using optical microscopy. The major distress observed was cracking, which was similar to the cracking of conventional concretes due to freeze/thaw damage. It was observed by several authors that PPs experienced localized failures caused by clogging, severe surface raveling, or cracking from inappropriate construction processes (Li et al., 2012). Chopra et al. (2011) carried out field and laboratory tests on PC pavements to evaluate their performance and determine some properties. Raveling caused by heavy vehicle loads and turning movements was observed soon after installation but decreased with time. Cracking was also observed; thus, it was recommended that PC should have a thickness greater than 6 in. to handle heavy vehicle loads. Brattebo & Booth (2003) examined the performance of four PP systems after 6 years of service. It was found that the structural integrity stayed constant with no major signs of wear reported. Nearly all storm water was infiltrated and contained lower levels of copper, zinc and motor oil compared to runoff from traditional asphalt pavements. It was concluded that the quantity and quality performance of the PP systems was still satisfying.

Existing PP systems were also studied to examine their hydrological performance. For Chopra et al. (2011), PC pavements can still have infiltration rates greater than the recommended minimum 2 in./hr. even under light to medium sediment accumulation conditions. Wanielista et al. (2007) evaluated the infiltration rates of eight PC pavements located in Florida. They concluded that those PC pavements with an average age of 12 years maintained proper infiltration rates even though no maintenance was performed at the sites. Forty-eight PPs including 15 concrete grid pavers (CGP), 14 PICP and 11 PC were tested by Bean (2005) to determine their infiltration rates pre- and post-maintenance. The CGP surfaces were filled with sand and PICP and PC were already constructed close to disturbed soil areas. Simulated maintenance was performed on CGP and consisted of removing manually 0.5 to 0.8 in. of top sand to simulate maintenance of a street sweeper. After maintenance, it was discovered that infiltration rates of the CGP improved significantly from 1.9 to 3.4 in./hr. Moreover, the median infiltration rates of PICP from sites with or without fines were 31 in./hr. and 800 in./hr., respectively. It was concluded that maintenance and appropriate location help maintain a high infiltration rate of PP surfaces (Bean, 2005). In North Carolina's Coastal Plain, a study was conducted by Bean et al. (2007) on four constructed PP applications to monitor and determine their effectiveness of reducing runoff quantity and improving water quality over a period of 10 to 26 months. The group of pavements studied was composed of 2 PICP, 1 PC, and 1 CGP. Only the PICP sites were monitored for water quality and it was found that the concentrations of nitrogen, ammonia, phosphorus, and zinc were lower in infiltrated storm water than traditional asphalt runoff.

The Minnesota Department of Transportation MnROAD conducted a research to study both the structural and hydrological performances of PP systems. PA and PC pavement test cells were constructed at the MnROAD research facility to simulate high load and low volume traffic, which helped evaluate their structural and hydrological performances (Kayhanian et al., 2015). As results, the lowest infiltration rate recorded was 0.6 in./s, high enough to handle large rainfall events. Infiltrated stormwater had constituents' concentrations that were within the acceptable range of water quality standards. The international roughness index (IRI) values passed the FHWA limit for an acceptable pavement. Some raveling and weathering were noticed after one year of testing. Some rutting was observed on the PA pavements and was much greater than conventional asphalt. The maximum surface deflections occurred in the summer and PC pavements deflected more than conventional concrete. No longitudinal or transverse cracking was observed after three years, probably due to the low stiffness compared to conventional pavements. The recorded reasons for clogging were particles from pavement raveling, silt particles transported by vehicles, and void content reduction due to heavy traffic loading. The PPs were vacuumed yearly using a Reliakor vacuum truck. It was concluded that there is no clear beneficial impact on infiltration rates. Therefore, it was recommended to vacuum twice a year (Kayhanian et al., 2015).

The effectiveness of maintenance practices for PP systems was also evaluated. Fitch & Bowers (2018) conducted a study to evaluate the performance of a PA pavement when it is monitored and maintained for 4 years using four protocols, which are "(1) no maintenance, (2) regenerative air vacuuming at 6-month intervals, (3) conventional vacuuming at 6-month intervals, and (4) regenerative air vacuuming at 12-month intervals". The results showed that the infiltration rate of the PA continued to decrease progressively over the course of the study despite the maintenance protocols. It was concluded that sedimentation might not be the real cause of permeability reduction in PP systems. It was also predicted that the PA will continue to function properly for a period of at least 12 years.

The longevity of a PP depends on the quality of installation, the surrounding conditions, and maintenance. The design life of permeable pavement ranges from 20 – 30 years for PC, 20 – 30 for PICP, and 15 – 20 for PA (Beisch & Foraste, 2013). However, Dylewski et al. n.d. stated that the lifespan of PA is approximately 30 years.

2.4 Challenges with Permeable Pavements

In the lifetime of PPs, complications or issues may occur and jeopardize the efficiency and durability of the pavements. For instance, a survey was done by Harvey J. et al. (2017) to ask 37 designers with PP experience to name the three most significant issues that affect the implementation of full PPs. The answers were summarized in Table 1.

Table 1. Issues affecting implementation of PP (adapted from Harvey et al., 2017)

Issues	% of contribution affecting PPs' implementation
Installation	11%
Unfamiliarity with design	8%
Public perception	3%
Maintenance	19%
Quality of construction	8%
Not strong enough to withstand traffic	3%
Maintaining native soil stability	3%
Conflict with utilities	8%
Non-compliance with current codes	3%
Higher cost	11%
Water ponding	8%
Poor mix design	3%
None so far	14%

Those complications are discussed below along with the various constraints and maintenance activities that should be considered prior, during, and after construction.

Structural damage: PPs are prone to serious damage in high traffic areas because of their lower strength compared to conventional pavements (Dylewski et al., n.d.). Thus, PPs should be built only for low traffic areas.

Reduced storage capacity: The stormwater storage capability is reduced, and base/subbase materials are shifted when PPs are constructed on steep slopes. Thus, the slope of the construction site should be between 1% to 5% (City of Birmingham, 2019). Contrary to City of Birmingham (2019), Dylewski et al. (n.d.) stated that slopes should not be greater than 2% and best management practices (BMPs) for erosion and sediment control should be considered if pavement construction starts upslope. Other researchers (Beisch & Foraste, 2013) state the maximum slope should be 1% with an underdrain.

Reduced infiltration rate: The infiltration of the soil can be reduced if unnecessary compaction of the soil occurs during construction. Underdrain is required if the native soil has a California Bearing Ratio (CBR) less than 4% because it may need to be compacted to at least 95% of the standard proctor density, thus reducing its infiltration rate (City of Birmingham, 2019; Beisch & Foraste, 2013). Stormwater cannot be drained properly when native soils have clay content higher than 20%, so hydrologic soil groups (HSG) C and D should be avoided (Dylewski et al., n.d.).

Surface clogging: Clogging or sediment buildup can prevent PPs from working properly, therefore regular inspections of the pavements and other contributing areas should be done to avoid potential pollutants (City of Birmingham, 2019). The frequency of maintenance must be increased if PPs are built around areas with vegetation (Dylewski et al., n.d.).

Groundwater contamination: Groundwater can be contaminated by stormwater pollutants if water table is too close to the base layer. A minimum distance of 2 ft for separation is recommended between the base layer and water table level (Beisch & Foraste, 2013; Dylewski et al., n.d.). In addition, PPs are not recommended to treat runoff from industrial or commercial areas to avoid contamination of groundwater because those areas may have high concentrations of soluble pollutants or pesticides (Dylewski et al., n.d.).

Other constraints: The presence of bedrock within 2 ft beneath the pavement surface is a problem that can be fixed solely by the use of underdrain. PPs should be built 100 ft away from water supply wells and 50 ft away from septic systems. Clearance of minimum 5 ft is required from down-gradient wet utility lines (City of Birmingham, 2019). Runoff coming from contributing impervious surface areas into PPs represents a large amount of total runoff volume. To limit that amount and avoid very thick reservoir layer, it is recommended that the ratio of impervious surface areas to PP area should not be greater than 2:1 (Dylewski et al., n.d.).

Maintenance / cleaning activities: Maintenance is an indispensable activity to assure an excellent performance and an extended lifespan of PPs. According to the City of Birmingham (2019), surface clogging caused by organic matter and sediment is the most common maintenance problem for PPs. The two most common pieces of maintenance equipment are mechanical street sweepers and regenerative air street sweepers, which remove surface particles by utilizing brushes and air, respectively (Dylewski et al., n.d.). However, those sweepers, according to Smith (2006), should not be used because they just move the sediment across the surface instead of removing it. The preferred equipment is the vacuum street sweeper because it is the most powerful and removes particles above and below the surface, but it is the most expensive.

According to the City of Birmingham (2019), vacuum sweeping should be done at least once a year or according to the rate at which sediment is deposited on the pavement surface. Soil's intrusion at the bottom of the reservoir layer is another issue that can be solved by the addition of geotextile or filter fabric. It can be difficult sometimes to access, inspect, and maintain PPs. For that, an observation well should be installed to facilitate periodic inspection and maintenance (City of Birmingham, 2019; Beisch & Foraste, 2013). Furthermore, a strong maintenance access road that is 20 ft wide and has a minimum drive path of 12 ft should be installed for access of maintenance heavy vehicles like street sweepers (Dylewski et al., n.d.). Slight raveling of PC particles is sometimes detected in the first few weeks after installation, but it should be negligible (Dylewski et al., n.d.).

Dylewski et al. (n.d.) suggested a maintenance schedule as shown in Table 2. More details concerning all the measures stated above can be found in the post construction storm water design manual for the city of Birmingham (City of Birmingham, 2019; Dylewski et al., n.d.).

Table 2. Maintenance schedule for permeable pavements (adapted from Dylewski et al., n.d.)

Task	How Often	Comments
Street Sweeping	Quarterly	Street sweeping will remove surface debris that can potentially clog the permeable pavement surface. Quarterly street sweeping is suggested, but increased frequency is recommended.
Inspection for Surface Deterioration	Quarterly	Inspections should be made once a quarter or following a 0.5” or greater rain event.
Inspect for Sediment	Monthly	Confirm that permeable pavement surface is free of sediment and debris.
Weed Removal	When they appear	Weeds should be eradicated using glyphosate. Hand pulling can disturb joint material in PICPs.
Mowing of Adjacent Land Areas	When needed	Clippings should be collected and removed from the site.
Stabilize Surrounding Land	When needed	Surrounding land should always be stabile to minimize sediment entry into the permeable pavement.

2.5 Existing Design Methods

The design of PPs is done through two major steps, which are the hydrological analysis where the reservoir storage depth is determined and the structural analysis where the thicknesses of the surface layer and/or the base layer are determined. The most conservative result from both methods controls the design (Weiss et al., 2017). On one hand, there are many hydrological design methods, but it is possible to choose one for all types of PP. On the other hand, there are also different structural design methods, but the chosen method is not the same for PC, PA or PICP (Weiss et al., 2017).

2.5.1 Hydrological Design Methods

During the hydrological design process, the reservoir thickness required to store stormwater runoff temporarily before it infiltrates the native soil is determined (Weiss et al., 2017). Thus, the permeability of the native soil is an important factor in the design process. In addition, the design runoff volume is also important because the storage capacity is designed according to the runoff volume anticipated. Even though this process is the same for all PPs, there is more than one method available, and no single method has been standardized. Since no method was chosen and imposed as standards, many organizations developed and proposed different methods with details (Weiss et al., 2017). A few of the common methods that have been used in the industry so far are discussed below.

The curve number method: According to Leming et al. (2007), the NRCS (National Resource Conservation Service) method or the curve number (CN) method is recommended for the hydrological design of PPs. The curve number method is used to estimate total runoff based on the soil or cover conditions of the site. The complete guide for this method is provided with details in (NRCS, 1986). The curve number method uses the 24 hours design storm and accounts for impervious areas and other surfaces. This allows for a complete analysis of the system through an empirical approach based on large datasets. Leming et al. (2007) suggested that the 2-year return period, 24 hours storm should be used to determine the storage depth and that the 10-year, 24 hours storm should be used for performance check. The curve number is determined from a table in NRCS (1986) using the cover conditions of the site and the hydrologic soil group, which can be found in USGA (2019). In NRCS (1986), hydrologic NRCS (HSG) A and B are considered best suited for PPs because they facilitate the infiltration of water. When a site contains various surface covers, the curve number can be estimated as a composite of curve numbers of smaller areas of the site. The curve number varies from 0 to 100 and the higher the curve number value, the less pervious the cover material is. For instance, a cover material with a curve number of 100 will have all rainfall behave as runoff on its surface while a cover material with a lower curve number will only have some of the rainfall behave as runoff. As described in NRCS (1986), the total depth of runoff is usually estimated using Equations 1 and 2.

$$Q = \frac{(P-0.2S)^2}{P+0.8S} \tag{1}$$

$$S = \frac{1000}{CN-10} \tag{2}$$

where, CN = curve number of the site, Q = total runoff depth (inches), S = the area or basin retention (inches), and P = precipitation (inches), which can be found in NOAA (2017).

The expected result from the curve number method should be the total runoff volume. According to NAPA (2008), the curve numbers for PP surfaces have not been determined yet, therefore, it was suggested to treat PPs as conventional dense-graded pavements with a curve number of 98. Various curve numbers were also recommended by agencies. For example, since nearly all soils within the city of Auburn are hydrologic soil group (HSG) B soils, it is recommended that a curve number of 85 should be used for PPs (City of Auburn, 2010). NAPA (2008) stated that the two most common methods to determine stormwater runoff are the curve number method and the rational method.

The rational method: According to Leming et al. (2007), this method is used to determine the maximum runoff rate or peak flow occurring at a specific time and location instead of the total amount of runoff. The rational method may be selected for PP design, but it is not as thorough as the curve number method. Leming et al. (2007) stated that the rational method should be used with caution to design for PPs because it might provide acceptable results, however all the advantages of PP systems may not be captured. In the rational method, the peak flow is obtained using Equation 3.

$$Q = CIA \tag{3}$$

where, Q = peak flow (ft^3/s), I = average rainfall intensity (in./hr), A= area of the watershed (acres), and C = runoff coefficient for the surface (ranges from 0 to 1). The higher the runoff coefficient value, more rainfall is expected to runoff the surface. The rational method utilizes short intense storms of 15 to 30 minutes duration for the analysis of small urban watershed. According to Weiss et al. (2017), the time of concentration of the watershed should be equal to the duration of the design storm and the return period of the storm may dictate the variation of the runoff coefficient. Leming et al. (2007) provided some estimates for the runoff coefficient values for PPs. The values vary from 0.05 to 0.35 depending on the infiltration rate of the native soil. Unlike Leming et al. (2007), NAPA (2008) does not recommend the use of the rational method for the hydrologic design of PPs because it can lead to interpretation problems.

The ICPI method: Another method called "the Interlocking Concrete Pavement Institute (ICPI) method" was introduced by Smith (2006) from the ICPI. This design method assumes that the total runoff infiltrates the PP and the underlying soil in a specific amount of time. This means that the pavement is considered 100% permeable when working properly. In this method, the catchment for the PP is composed of the surface area being analyzed and any other area that contributes runoff to it. Therefore, the base/subbase layer is sized to store the total runoff volume from the analyzed pavement area and the adjacent contributing areas. Since the NRCS method recommends the use of 24-hour storm events for the hydrologic design, this design method aims to control the increased runoff for a specific 24-hour storm. The storm return period and the duration should be provided by the locality, but a first flush event must be the minimum selected especially when the increase in peak flow cannot be managed. The NRCS Type II storm is recommended and the time for the stormwater to fill the base layer should not be more than 2 hours. In the ICPI method, the first step is to determine the total runoff volume based on the rainfall precipitation and the runoff volume from contributing areas. Next, the depth of the aggregate base is calculated using Equation 4.

$$d_p = \frac{\Delta Q_c R + P - fT}{V_r} \tag{4}$$

where, d_p = depth of the base/subbase layer, ΔQ_c = runoff depth from watershed flowing on to PP area, P = depth of rainfall, f = subgrade final infiltration rate, T = effective time to fill the base/subbase layer, V_r = void ratio of aggregate base/subbase layer, and R = the ratio of the contributing area and the PP area (A_c/A_p), where A_c = contributing area, and A_p = PP area. Finally, the maximum allowable depth of the base/subbase layer or reservoir layer is determined using Equation 5.

$$d_{max} = f \times \frac{T_S}{V_r} \tag{5}$$

where, d_{max} = maximum allowable depth of base/subbase layer, f = subgrade final infiltration rate (it is the most important parameter in the design of PPs with a minimum value of 10^{-5} cm/s (Chai et al., 2012)), V_r = void ratio of aggregate base/subbase layer, and T_S= maximum storage time. The depth of the reservoir layer d_p must be less or equal to the maximum allowable reservoir depth d_{max}. The PP area can be increased, or a lower design storm can be selected when d_p does not satisfy the criteria. The City of Birmingham (2019) suggested the same method for the sizing of PP; however, it was explained in a more detailed manner. In the situation where d_p is greater than d_{max}, the use of underdrains is recommended.

<u>The Los Angeles county method:</u> This method is also a hydrological design method that the American Concrete Pavement Association (ACPA) chose to use in their software, PerviousPave. This software can be used to design a PC pavement system. In the Los Angeles county method, the thickness of the reservoir layer can be adjusted as necessary until all the requirements for stormwater management are met (Rodden & Smith, 2011). This method is similar to the ICPI method and both yield approximately the same results. According to Rodden & Smith (2011), this method is an adaptation of the method provided by the department of public works in the county of Los Angeles. The depth of the reservoir layer is obtained using Equation 6.

$$h_s = \frac{1}{r_s}\left(\frac{12\times V}{A_p} - h_{curb} - r_c \times h_c\right) \tag{6}$$

where, h_s = depth of the reservoir layer (inches), r_s = void ratio of reservoir layer (%), V = total volume of water (ft^3), A_p = area of PC pavement (ft^2), h_{curb} = height of curb or allowable ponding height (inches), r_c = void ration of PC pavement (%), and h_c = height of PC pavement (inches). The detention time of water is also calculated using Equation 7 to verify that it does not exceed the desired detention time chosen by the designer (usually 24 hours).

$$t_d = \frac{12 \times V}{A_p \times E} \tag{7}$$

where, t_d = detention time (hours), and E = permeability of the native soil (in./hr).

Some of these methods are built in hydrological computer modeling tools. All methods stated above are eligible to use for hydrological design of PPs. However, the most comprehensive method is the ICPI method because it is the only method that yields results concerning the total volume runoff and the depth of the reservoir layer (Weiss et al., 2017). The curve number method and the rational method can be used, but the lack of accurate curve number values and runoff coefficient values can lead to low accuracy problems.

2.5.2 Structural Design Methods

MEPDG: For PC pavements, the American Concrete Pavement Association (ACPA) developed a structural design process based on the Mechanistic-Empirical Pavement Design Guide (MEPDG) method (ACPA, 2020). The mechanistic-empirical design is a combination of mechanical modeling elements and performance observations to determine the required thickness of a pavement using a given set of design inputs (Gedafa et al., 2011). This design method was incorporated in their software called PerviousPave. The specific model employed by PerviousPave is the Westergaard's model of a plate on a Winkler foundation, which is originally used for conventional rigid pavements (Weiss et al., 2017). With the help of falling weight deflectometer data obtained from testing on numerous PC pavements, Vancura et al. (2011a) and AlShareedah & Nassiri (2019) demonstrated that Westergaard's model is also applicable on PC pavements. According to Rodden & Smith (2011), the method developed by ACPA was introduced in 2010 and is an adaptation of the design methodology used in the software called StreetPave. In StreetPave, this method consisted of both erosion and fatigue analyses of concrete pavements (Titus-Glover et al., 2005). Before the incorporation of this method in PerviousPave, the erosion failure criterion was excluded because the materials used for the reservoir layer of PC pavements are non-erodible (Rodden & Smith, 2011). The fatigue model is the only criterion in this method and consists of establishing the number of allowable load repetitions for a given stress ratio. Equation 8 is used to determine the allowable load applications until failure occurs (Titus-Glover et al., 2005; AlShareedah & Nassiri, 2017).

$$Log\ N_f = \left[\frac{-SR^{-10.24}\, log(1-P)}{0.0112}\right]^{0.217} \tag{8}$$

where, N_f = allowable load applications, SR = stress ratio (%), and P = probability of failure (%). For each axle type, the fatigue damage is calculated using the Miner's damage hypothesis equation shown in Equation 9 (Rodden & Smith, 2011).

$$FD = \frac{N}{N_f} \tag{9}$$

where, N = number of load applications obtained from user traffic data. The American Concrete Institute (ACI, 2008), in the ACI 330 guide, provided traffic categories with axle loads to use for design of concrete parking lots. Huang (2004) also provided some formulas to obtain the design traffic for loads calculations. The loads can be used to determine the number of load applications for each axle type. The total fatigue damage is obtained using Equation 10 (Rodden & Smith, 2011).

$$FD_{total} = FD_{single} + FD_{tandem} + FD_{tridem} \tag{10}$$

where, FD_{total} = total fatigue damage (%), FD_{single} = fatigue damage due to single axle loads (%), FD_{tandem} = fatigue damage due to tandem axle loads (%), FD_{tridem} = fatigue damage due to tridem axle loads (%). The minimum required thickness of the PC pavement is found when the total fatigue damage reaches 100%, which is the limiting criterion of the structural design.

The composite modulus of subgrade/subbase reaction, which is also one important component of the design method was not discussed by Rodden & Smith (2011) in their guidelines. The formula to determine that value can be found in (Barker & Alexander, 2012).

AASHTO 1993: For PA pavements and PICP, the design guidelines called AASHTO 1993 published by the American Association of State Highway and Transportation Officials (AASHTO) are commonly used. The 1993 AASHTO guide is an empirical methodology originally based on algorithms developed from the AASHO road test in the late 1950's and 1960's (Gedafa et al., 2011). The AASHO road test was performed under climatic setting, pavement materials, subgrade soils, loading characteristics that were specific and accurate for that period. Therefore, the results from that test may not be representative of today's conditions and can lead to a degree of uncertainty about the methods. However, Schwartz & Hall (2018) and Smith (2012) recommended the AASHTO 1993 method for PA and PICP, respectively. The principal equation for the AASHTO pavement structural design method is shown in Equation 11.

$$logW_{18} = Z_R S_0 + 9.36\log(SN+1) - 0.20 + \frac{log\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.4+\frac{1094}{(SN+1)^{5.19}}} + 2.32logM_R - 8.07 \qquad (11)$$

where, W_{18} = design traffic loading (quantified in terms of 18-kips equivalent single axle loads (ESALs)). Schwartz & Hall (2018) provided values of W_{18} for PA ranging from 3,000 ESALs for residential driveways to 1,100,000 ESALs for roads supporting heavy trucks daily. Z_R = standard deviation associated with reliability level. Schwartz & Hall (2018) recommended reliability values for PA ranging from 50% for local driveways to 99.9% for interstate freeway. Smith (2012) stated that the 80% should be used for reliability level. S_0 = standard deviation. For Schwartz & Hall (2018) and Smith (2012), 0.45 is the appropriate standard deviation value for PA and PICP. ΔPSI = allowable change in present serviceability index and 1.7 is the recommended value (Schwartz & Hall, 2018). ΔPSI = P_i - P_t where, P_i = initial serviceability and P_t = terminal serviceability. MR = subgrade resilient modulus (kPa). Finally, SN = structural number of the pavement where, SN = $\Sigma a_i d_i$, a_i = structural layer coefficient, d_i = layer thickness. To obtain the pavement layer thickness easily, an Excel spreadsheet can be created using the AASHTO formula.

According to Weiss et al. (2017), the ACPA design method developed for PerviousPave is an acceptable method for PC pavements. They suggested that additional modifications continue to be made as more performance data for PC pavements become available. Concerning PA and PICP, AASHTO 1993 is still an appropriate method for their design. However, there is urgent need for a mechanistic-empirical approach for more adapted results (Weiss et al., 2017).

2.6 Costs of Permeable Pavement Components

PA is more cost effective than PC and PICP. Costs of PA are at least 10% higher that of conventional asphalt. The Water and Environmental Research Foundation (WERF) estimated that the cost of PA ranges between \$0.5 to \$1/ft^2 (2005 figures) and the University of New Hampshire Stormwater Center (UNHSC) estimated that to be around \$2.80/ft^2 (Beisch & Foraste, 2013). The costs of PP components in 2005 dollars, according to WisDOT (2012), are \$0.5 - \$1/ft^2 for PA, \$2 - \$7/ft^2 for PC, \$30 - \$35/yd^3 for aggregate, \$0.7 - \$1/ft^2 for geotextile fabric, and \$8 - \$10/yd^3 for excavation. The costs of the aggregate, geotextile fabric and excavation are the same for all types of PPs (WisDOT, 2012).

Rehan et al. (2018) provided significant cost data for construction, maintenance, and stormwater treatment of each type of PP as well as conventional hot-mix asphalt (HMA) and Portland cement concrete (PCC) pavements. They conducted a life cycle cost analysis for four types of pavement, which are PA, PC, HMA and PCC. All costs were in 2018 dollars and were discounted to present worth using 4% discount rate. PA was found to be the most economical for the 20-, 30- and 40-year analysis period. Walsh & Smallridge (2001) estimated the cost of interlocking concrete pavers to be equal to \$3.02/ft^2 (2001 dollars) for a 4 in. thick paver. Wood & Volkert (2019) provided the cost for excavation to be equal to \$15/yd^3 (2019 dollars).

Since costs are usually provided for different locations and time periods, Olson et al. (2017) provided formulas to adjust costs for inflation and location. Those formulas use the construction cost index history in Engineering News Record (ENR 2021).

2.7 Existing Tools and Cost Optimization Practices

The American Concrete Pavement Association (ACPA) created a software called PerviousPave that has for objective the structural and hydrological design of PC pavements. The tool relies on a fatigue model as failure criterion for structural design and uses the Los Angeles County method for hydrological design (Rodden & Smith, 2011). The limitation of the tool is that it focuses on only one type of PP, which is PC.

Olson et al. 2017 developed a spreadsheet tool to evaluate the effectiveness and life cycle costs of Best Management Practices (BMP) such as PPs. This tool uses construction costs, land costs, maintenance costs, rehabilitation or replacement costs, and administrative costs of BMP components. However, this tool does not take into account the design of PPs and their optimization.

As an optimization practice for PP, Beisch & Foraste (2013) recommended combining PP with conventional pavement and limiting the external contributing drainage area to PP ratio to 2:1. According to them, this will reduce the square footage of PP and will result in direct cost savings.

CHAPTER 3: METHODOLOGY

In this chapter, the processes or methods used to achieve the research goals are explained with details. The main objective of the research was to create a user-friendly tool to design and optimize costs of PC, PA, and PICP. The design of the cross section for each type of PP consists of two major steps, the hydrologic and the structural design. Generally, except for PA, the structural integrity of the pavement is checked first then follows the hydrologic design to verify that the base/subbase of the pavement can effectively store the expected water volume. A unique hydrologic design process can be adopted for all types of PP whereas various structural design methods were suggested by agencies and associations (Weiss et al., 2017). The construction costs used in the tool were obtained through literature. The algorithms for design and cost optimization of PPs were simply written in the Excel tabs of the tool using Excel functions. This was done to ensure clear understanding of the user and easy access for enhancement of the tool in the future to include more GIs. Additionally, Visual Basic Applications (VBA) was used as appropriate to enhance the user experience by assigning macro color codes to control buttons. Figure 6 is a flow chart diagram that shows how the developed tool operates.

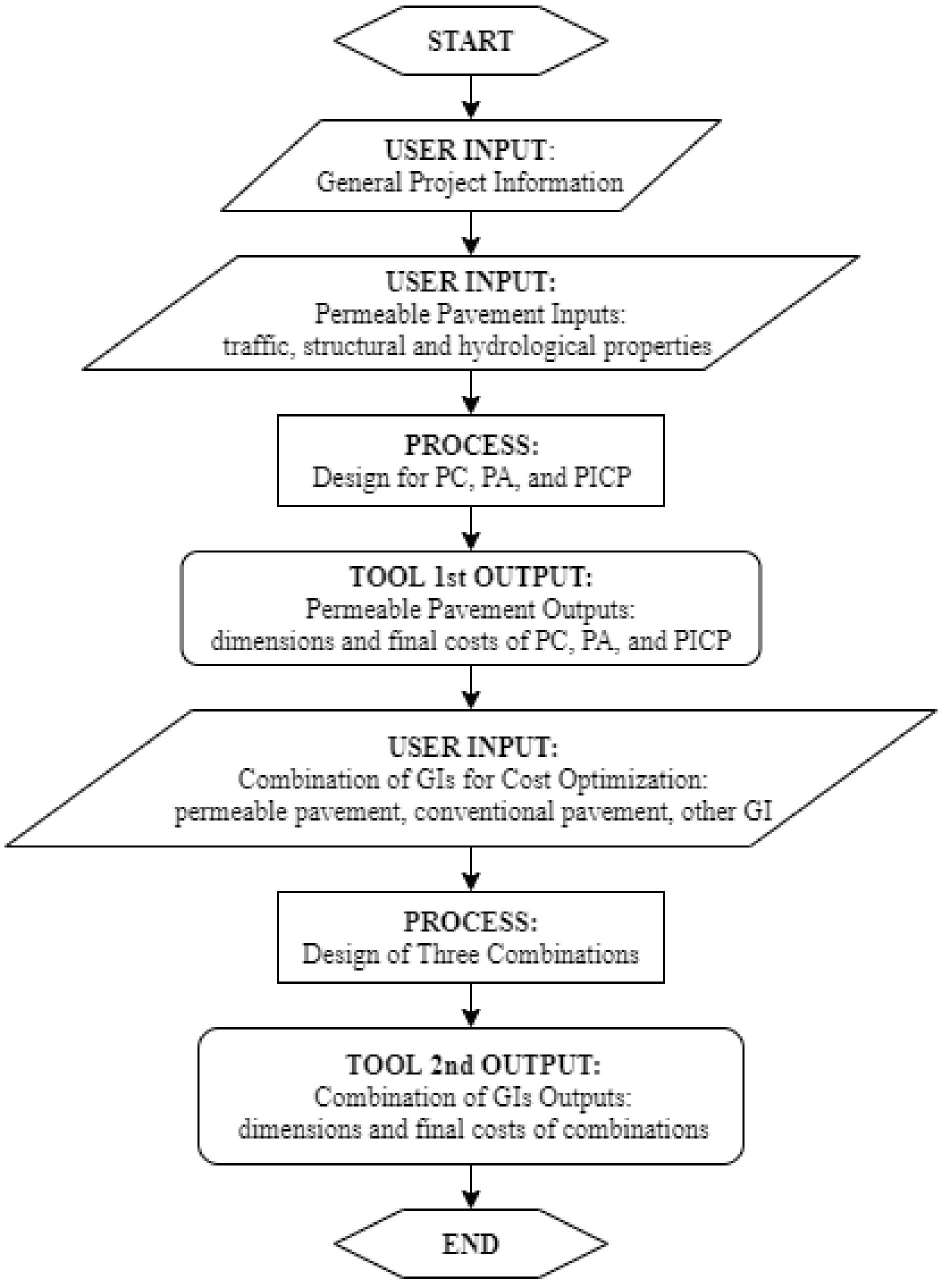

Figure 6. Process flow diagram for the COTPP

3.1 Hydrological Design

The purpose of the hydrologic design process is to calculate the required depth of the reservoir layer to store temporarily and evacuate storm water runoff within a desired time. The PP layer, curbs, and the base/subbase layer can all constitute the reservoir (Weiss et al., 2017).

However, for design purposes and to facilitate cost calculations, only the base/subbase layer was used for water storage capacity in the COTPP. The base/subbase layer is the main stormwater storage and using the PC surface layer as part of the storage can accelerate the degradation of the pavement with time. Since no method was chosen and imposed as standards, many organizations developed and proposed various methods, sometimes detailed, which can be used to design PPs. A few of the common methods that have been used in the industry so far were reported by Weiss et al. (2017) and are: the curve number method; the rational method; the Interlocking Concrete Pavement Institute (ICPI) method or the permeable interlocking concrete pavers (PICP) method; the Los Angeles county method; and computer modelling. Because "it is the most comprehensive method" according to Weiss et al. (2017), the ICPI method is the hydrological method used in the COTPP. The steps of this method are well detailed in the manual provided by the City of Birmingham (2019); therefore, those steps were the ones chosen for the COTPP.

3.1.1 The ICPI Method

The process followed by the tool to determine the depth of the reservoir for each type of PP is explained below in term of steps.

***Step 1*: Determine the treatment volume of water to be drained by the PP using Equation 12 (City of Birmingham, 2019).

$$Tv = Rv \times (Ap + Ac) \times P \tag{12}$$

where, T_v = treatment volume of water (ft^3), R_V = volumetric runoff reduction coefficient (R_V = 0.95 for impervious areas), A_p = pervious area or permeable pavement area (ft^2), A_c = contributing impervious area (e.g., roofs, hardscapes, etc.) (ft^2), and P = rainfall depth or precipitation (ft), which can be found on the National Oceanic and Atmospheric Administration (NOAA) website.

***Step 2*:** Calculate the depth of the reservoir layer with no underdrain via Equation 13 (City of Birmingham, 2019; Smith, 2006; Weiss et al., 2017).

$$d_p = \frac{\left\{(d_c \times R) + P - \left(\frac{i}{2} \times t_f\right)\right\}}{V_r} \tag{13}$$

where, d_p = depth of reservoir layer (ft), d_c = (T_v/A_c) = runoff's depth from the contributing impervious area (ft) (permeable paving surface is not included), R = A_c/A_p = ratio of the impervious area (A_c) to the pervious area (A_p), P = rainfall depth (ft) which can be found on the NOAA website, i = infiltration rate or permeability of native soil (ft/day). The infiltration rate was divided by a safety factor of 2 as recommended by the City of Birmingham (2019) for design purposes. The minimum acceptable infiltration rate is 0.5in./hr (Dylewski et al. n. d.; City of Birmingham, 2019). t_f = time to fill the reservoir layer (day) (typically 2 hours or 0.08333 day), and V_r = void ratio of the reservoir layer (typically 40% or 0.4).

***Step 3*:** Calculate the maximum depth of the reservoir layer using Equation 14 (City of Birmingham, 2019; Smith, 2006; Weiss et al., 2017).

$$d_{p-max} = \frac{\left(\frac{i}{2} \times t_d\right)}{V_r} \tag{14}$$

where, d_{p-max} = maximum depth of the reservoir layer (ft), i = infiltration rate or permeability of native soil (ft/day). It is divided by a safety factor of 2 for design purposes. The minimum acceptable infiltration rate is 0.5 in./hr. t_d = maximum allowable time to drain the reservoir layer (typically 1 to 2 days). In the COTPP, 24 hours (1 day) was used as recommended by the city of Birmingham (2019) and Dylewski et al. (n. d.), V_r = void ratio for the reservoir layer (typically 40% or 0.4). The use of underdrain is recommended if d_p is greater than d_{p-max} (City of Birmingham, 2019).

3.2 Structural Design

Unlike the hydrologic design process, each type of PP has a unique structural design method because of the lack of a standardized method.

3.2.1 Pervious Concrete

For the structural design of PC, a mechanistic design methodology was adopted by the American Concrete Pavement Association (ACPA). The ACPA developed a software called PerviousPave that uses a design methodology that was originally created for jointed plain concrete pavements and used in a software called StreetPave. Despite the fact that the general criteria for most jointed plain concrete pavement are both fatigue (cracking) and erosion (faulting or surface smoothness), PerviousPave uses fatigue as the sole failure criterion because it has not been proven yet that erosion occurs in PC pavements (Rodden & Smith, 2011). The design method used in PerviousPave was adopted in the COTPP to determine the thickness of PC following the steps below:

<u>Step 1</u>**:** Estimate the composite modulus of subgrade/subbase reaction using Equation 15 (Barker & Alexander, 2012).

$$\ln(k) = -2.807 + 0.1253(lnD_{SB})^2 + 1.062(lnM_R) + 0.1282(lnD_{SB})(lnE_{SB}) - 0.4114(lnD_{SB}) - 0.0581(lnE_{SB}) - 0.1317(lnD_{SB})(lnM_R) \quad (15)$$

where, k = composite modulus of subgrade/subbase reaction (pci), D_{SB} = thickness of the subbase or minimum reservoir depth (in.), E_{SB} = elastic modulus of the subbase layer (psi), which according to Schwartz & Hall (2018) should vary from 15000 psi to 45000 psi, and M_R = resilient modulus of the subgrade (psi).

<u>Step 2</u>**:** Calculate the radius of relative stiffness via Equation 16 (Rodden & Smith, 2011).

$$L = \sqrt[4]{\frac{E \times h_c^3}{12(1-u^2)k}} \tag{16}$$

where, L = radius of relative stiffness (in.), E = modulus of elasticity of the PC (psi), k = composite modulus of subgrade/subbase reaction (pci), μ = poisson's ratio of the concrete - typically 0.15, and h_c = thickness of PC layer (in.) (typically less than 12 in.).

***Step 3*:** Calculate the adjustment factor for the effect of axle loads and contact area using Equations 17 through 19 (Rodden & Smith, 2011).

$$f_1 = \begin{cases} \left(\frac{SAL}{24}\right)^{0.94} \times \frac{24}{18} \quad \text{for single axles} & (17) \\ \left(\frac{TAL}{48}\right)^{0.94} \times \frac{48}{36} \quad \text{for tandem axles} & (18) \\ \left(\frac{TRIAL}{72}\right)^{0.94} \times \frac{72}{54} \quad \text{for tridem axles} & (19) \end{cases}$$

where, SAL = single axle load (kips), TAL = tandem axle load (kips), and TRIAL = tridem axle load (kips).

***Step 4*:** Calculate the adjustment factor for a slab with no concrete shoulder using Equations 20 through 21 (Rodden & Smith, 2011).

$$f_2 = \begin{cases} = 0.892 + \left(\frac{h_c}{85.71}\right) - \frac{h_c^2}{3000} \quad \text{for no shoulders} & (20) \\ = 1 \quad \text{for with shoulder} & (21) \end{cases}$$

where, h_c = thickness of PC layer (in.). For parking lots applications, there is usually no shoulder.

Step 5: Estimate the adjustment factor to account for the effect of truck (wheel) placement at the edge of the PC slab (Rodden & Smith, 2011).

f_3 = typically assumed as 0.894 for 6 percent trucks at the slab edge, which represents a conservative estimate for applications such as parking lots because there is not as much traffic as on street roads or highways.

Step 6: Calculate the adjustment factor to account for roughly 23.5% increase in concrete strength with age after the 28th day and reduction of one coefficient of variation (COV) to account for materials variability using Equation 22 (Rodden & Smith, 2011).

$$f_4 = \frac{1}{[1.235 \times (1 - \mathrm{COV})]} \tag{22}$$

Step 7: Determine the equivalent moments via Equations 23 through 28 (Rodden & Smith, 2011).

$M_e =$

$-1600 + 2525 \times \log(L) + 24.42 \times L + 0.204 \times L^2$ (for single axles with no edge support) (23)

$3029 - 2966.8 \times \log(L) + 133.69 \times L - 0.0632 \times L^2$ (for tandem axles with no edge support) (24)

$-414.6 + 1460.2 * \log(L) + 18.902 * L - 0.1243 * L^2$ (for tridem axles with no edge support) (25)

$(-970.4 + 1202.6 \times \log(L) + 53.587 \times L) \times (0.8742 + 0.01088 \times k^{0.447})$ (for single axles with edge support) (26)

$(2005.4 - 1980.9 \times \log(L) + 99.008 \times L) \times (0.8742 + 0.01088 \times k^{0.447})$ (for tandem axles with edge support) (27)

$(-88.54 + 134.0 \times \log(L) + 0.83 \times L) \times (11.3345 + 0.2218 \times k^{0.448})$ (for tridem axles with edge support) (28)

where, M_e = equivalent moment (psi), and L = radius of relative stiffness (in.)

For parking lot applications, there is usually no edge support.

Step 8: Determine the equivalent stress using Equation 29 (Rodden & Smith, 2011).

$$\sigma_{eq} = \frac{6 * M_e}{h_c^2} * f_1 * f_2 * f_3 * f_4 \tag{29}$$

where, σ_{eq} = equivalent stress (psi), M_e = equivalent moment (psi), h_c = thickness of PC layer (in.), and (f_1, f_2, f_3, f_4) = adjustment factors.

Step 9**:** Calculate the stress ratio through Equation 30 (Rodden & Smith, 2011).

$$SR = \frac{\sigma_{eq}}{f_r} \tag{30}$$

where, SR = stress ratio, f_r = flexural strength of the concrete (psi), and σ_{eq} = equivalent stress (psi).

Step 10**:** Estimate the probability of failure through Equation 31 (Rodden & Smith, 2011).

$$P = 1 - R \times \frac{SC}{50} \tag{31}$$

where, P = probability of failure (%), R = reliability (%), SC = percent slabs cracked at the end of pavement's life (%) – typically assumed as 15%. SC is a measure of pavement distress caused by fatigue damage in the slabs.

Step 11**:** Determine the allowable load applications to failure (N_f) using Equation 32 (Rodden & Smith, 2011).

$$\text{Log } N_f = \left[\frac{-SR^{-10.24}\log(1-P)}{0.0112}\right]^{0.217} \tag{32}$$

where, N_f = allowable load applications to failure for each axle type, SR = stress ratio, and P = probability of failure.

Step 12**:** Estimate the number of load application (N) for each axle type (Rodden & Smith, 2011). This can be calculated from the project's traffic data. However, the American Concrete Institute provided traffic data shown below in Figure 7 to represent the traffic data for different project applications or traffic categories. The categories are as follows: Category A is for car parking areas and access lanes; Category B is for shopping center entrance and service lanes, city and school buses parking areas and interior lanes, truck parking areas; Category C is for entrance and exterior lanes and truck parking areas; and Category D is for truck parking areas.

Axle load, kN (kips)	Axles per 1,000 trucks (excluding all two-axle, four-tire trucks)			
	Category A	Category B	Category C	Category D
18 (4)	1,693.31	1,693.31	—	—
27 (6)	732.28	732.28	—	—
36 (8)	483.10	483.10	233.60	—
44 (10)	204.96	204.96	142.70	—
53 (12)	124.00	124.00	116.76	—
62 (14)	56.11	56.11	47.76	—
71 (16)	38.02	38.02	23.88	1,000
80 (18)	—	15.81	16.61	—
89 (20)	—	4.23	6.63	—
98 (22)	—	0.96	2.60	—
107 (24)	—	—	1.60	—
116 (26)	—	—	0.07	—
Tandem axles				
18 (4)	31.90	31.90	—	—
36 (8)	85.59	85.59	47.01	—
53 (12)	139.30	139.30	91.15	—
71 (16)	75.02	75.02	59.25	—
89 (20)	57.10	57.10	45.00	—
107 (24)	39.18	39.18	30.74	—
125 (28)	68.48	68.48	44.43	—
142 (32)	69.59	69.59	54.76	2,000
160 (36)	—	4.19	38.79	—
178 (40)	—	—	7.76	—
196 (44)	—	—	1.16	—

Source: Data from ACI (2008).

Figure 7. Axle load distribution factors for different traffic categories (AlShareedah & Nassiri, 2019)

- Growth factor, design traffic, and number of load applications for each axle type expressed as Equations 33 through 35 (Huang, 2004):

$$\mathrm{GF} = \frac{(1+g)^n - 1}{g} \tag{33}$$

where, GF = growth factor, g = annual truck traffic growth or growth rate (%), and n = pavement design life (years).

$$Design\ Traffic = GF \times 365 \times ADTT \tag{34}$$

where, ADTT = average daily truck traffic, one-way (number of trucks), and GF = growth factor.

$$N = \Sigma\left(\frac{Axles}{1000\ trucks}\right) \times Design\ traffic \tag{35}$$

where, N = number of load applications for each axle type (single, tandem, and tridem axles), Axles/1000 trucks = number of axles for each axles type obtained from Figure 6, Design traffic = number of trucks during pavement design life.

***Step 13*:** Determine the fatigue damage for each axle type using Equation 36 (Rodden & Smith, 2011).

$$FD = \frac{N}{N_f} \tag{36}$$

where, FD = fatigue damage for each axle type, N = number of load applications for each axle type, and N_f = allowable load applications to failure for each axle type. The axle types are single, tandem and tridem.

***Step 14*:** Determine the total fatigue damage via Equation 37 (Rodden & Smith, 2011).

$$FD_{total} = FD_{single} + FD_{tandem} + FD_{tridem} \tag{37}$$

where, FD_{total} = total fatigue damage (%), FD_{single} = fatigue damage due to single axle loads (%), FD_{tandem} = fatigue damage due to tandem axle loads (%), FD_{tridem} = fatigue damage due to tridem axle loads (%).

The design criterion is satisfied when the total fatigue damage (FD_{total}) reaches 100% (or 1). Therefore, the thickness of the PC layer is increased incrementally in the process until 100% total damage is reached.

3.2.2 Porous Asphalt

The design guidelines for flexible pavements provided by AASHTO are usually used for the design of PA (Weiss et al., 2017). As a result, NAPA released some guidelines for the design of PA based on the AASHTO 1993 design method. After obtaining the reservoir thickness from the hydrological design, the following steps are taken to calculate the thickness of the PA layer:

***Step 1*:** Estimate the design traffic (W18) (Schwartz & Hall, 2018).

In the AASHTO 1993 design guide, traffic loading is quantified in terms of 18-kip ESALs. Even though most agencies have a system already setup to determine the design ESALs, that system or method may not be appropriate for PA since it is based on the effect of traffic loads on dense-graded hot-mix asphalt. According to Schwartz & Hall (2018), there is an alternative method, which is a traffic classification scheme for quantifying ESALs for lightly trafficked pavement such as PA. The traffic classification is summarized in Figure 8 where the corresponding ESAL is a function of traffic class and design period.

Type of facility and vehicle types	Maximum trucks/month (one lane)	Traffic class	Design period (years)	Design ESALs
Residential driveways, parking stalls, parking lots for autos and pickup trucks.	<1	Class I	5	3,000
			10	3,000
			15	5,000
			20	7,000
Residential streets without regular truck traffic or city buses; traffic consisting of autos, home delivery trucks, trash pickup, occasional moving vans, etc.	60	Class II	5	7,000
			10	14,000
			15	20,000
			20	27,000
Collector streets, shopping center delivery lanes; up to 10 single-unit or 3-axle semi-trailer trucks per day or equivalents; average gross weights should be less than the legal limit.	250	Class III	5	27,000
			10	54,000
			15	82,000
			20	110,000
Heavy trucks; up to 75 fully loaded 5-axle semi-trailer trucks per day; equivalent trucks in this class may include loaded 3-axle and 4-axle dump trucks, gross weights over 40,000 lbs.	2200	Class IV	5	270,000
			10	540,000
			15	820,000
			20	1,100,000

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Figure 8. Design ESALs form Traffic classification (Schwartz & Hall, 2018)

Step 2**:** Estimate the reliability (R or Z_R), standard deviation (S_0), and allowable change in present serviceability index (ΔPSI) (Schwartz & Hall, 2018).

According to Schwartz & Hall (2018), the use of existing policies for conventional flexible pavements in the selection process of design values of reliability (R), standard deviation (S_0) and allowable performance (ΔPSI) is appropriate for PA.

The recommendations from AASHTO 1993 for flexible pavements are as follows:

- For Reliability and standard normal deviate (R and Z_R), see Tables 3 and 4:

Table 3. Reliability recommendations (adapted from AASHTO 1993)

Functional Class	Recommended Reliability	
	Urban	Rural
Interstate Freeway	85 – 99.9%	80 – 99.9%
Principle Arterials	80 – 99%	75 – 95%
Collectors	80 – 95%	75 – 95%
Local	50 – 80%	50 – 80%

According to Schwartz & Hall (2018), the typical agency criteria for reliability is 75%.

Table 4. AASHTO standard normal deviate (ZR) values corresponding to selected levels of reliability (adapted from AASHTO 1993)

Reliability, %	50	75	80	85	90	95	99.9
Z_R	0.000	-0.674	-0.842	-1.036	-1.282	-1.645	-3.090

- For Standard Deviation (S_0): The recommended value is 0.45 (Schwartz & Hall, 2018). Weiss et al. (2017) stated that 0.44 is reasonable for flexible pavements and pavers.
- For Allowable Change in PSI as expressed as Equation 38 (Schwartz & Hall, 2018):

$$\Delta \mathrm{PSI} = \mathrm{P}_0 - P_t \tag{38}$$

where, P_0 = initial serviceability – 4.2 is assumed, and P_t = final serviceability, which is provided in Table 5 below:

Table 5. Recommended terminal serviceability values (adapted from AASHTO 1993)

Traffic Level, AADT	Terminal Serviceability, P_t
High: >10,000	3 to 3.5
Medium: 3,000 – 10,000	2.5 to 3
Low: <3,000	2 to 2.5

According to Schwartz & Hall (2018), the typical agency criteria for ΔPSI is 2.5. However, it would be more logical if P_t was equal to 2.5 since PPs support less traffic and Weiss et al. (2017) stated that P_t = 2.5 is reasonable for shoulder pavements. This would make ΔPSI = 4.2 – 2.5 = 1.7.

Step 3: Estimate the subgrade resilient modulus (M_R) (Schwartz & Hall, 2018).

According to Schwartz & Hall (2018), the resilient modulus for subgrade under conventional flexible pavements should be reduced by 25-50% to obtain the resilient modulus of subgrade under PA. This procedure is important because subgrades under PA pavements are not compacted and allow water to infiltrate, which means higher moisture content and lower strength. Some typical values of subgrade resilient modulus for PA pavements as a function of soil type are provided at the far right of Figure 9. It is important to note that there are other methods to calculate the subgrade resilient modulus. In the COTPP, the value can be entered as input or calculated from the soil California Bearing Ratio (CBR).

Soil Type	Unified Soil Class	Percent Finer Than 0.02 mm	Permeability	Frost Potential[1]	Typical CBR[2]	Design Class	Typical Minimum Flexible Pavement M_R (psi)[2]	Recommended Minimum Porous Asphalt Pavement M_R (psi)[2]
Gravels, crushed stone Little or no fines <0.02mm	GW,GP	0–1.5	Excellent	NFS	17	Very Good	20,000	20,000
Sands, sand-gravel mix Little or no fines <0.02mm	SW,SP	0–3	Excellent	NFS	17	Very Good	20,000	20,000
Gravels, crushed stone Some fines <0.02mm	GW,GP	1.5–3	Good	PFS	17	Very Good	20,000	20,000
Sands, sand-gravel mix Some fines <0.02mm	SW,SP	1.5–3	Good	PFS	17	Very Good	20,000	20,000
Gravelly soils Medium fines <0.02mm	GW,GP,GM	3–6	Fair	Low	8	Good	12,000	9,000
Sandy soils Medium fines <0.02mm	SW,SP,SM	3–6	Fair	Low	8	Good	12,000	9,000
Silty gravel soils High fines <0.02mm	GM GW-GM,GP-GM	6–10 10–20	Fair to Low	Medium	8	Good	12,000	9,000
Silty sand soils High fines <0.02mm	SM SW-SM,SP-SM	6–15	Fair to Low	Medium	8	Good	12,000	9,000
Clayey gravel soils High fines <0.02mm	GM,GC	>20	Fair to Low	Medium to High	5	Medium	7,500	3,750
Clayey sand soils High fines <0.02mm	SM,SC	>20	Low to Very Low	Medium to High	5	Medium	7,500	3,750
Very fine silty sands	SM	>15	Low	High to Very High[3]	5	Poor	7,500	3,750
Clays, PI>12	CL,CH		Very Low	High[3]	3	Poor	4,500	2,250
All silt soils	ML,MH		Very Low	High to Very High[3]	3	Poor	4,500	2,250
Clays, PI<12	CL,CL-CM		Very Low	High to Very High[3]	3	Poor	4,500	2,250
Other fine-grained sols	OL		Very Low	High to Very High[3]	<3	Very Poor	3,000	1,500
Highly organic soils	OH		Very Low	Replace		Replace		

Figure 9. Typical values for resilient modulus of subgrade under porous asphalt (Schwartz & Hall, 2018)

***Step 4*:** AASHTO layer coefficients (a_i) (Schwartz & Hall, 2018).

- For porous asphalt:

  According to Schwartz & Hall (2018), the most widely used layer coefficient for dense graded asphalt concrete is $a_1 = 0.44$, which is an average value derived from the AASHO Road Test. However, NAPA recommended to use 0.40 for PA surface layer.

- For Stone Recharge Bed (Coarse Aggregate Base):

  NAPA recommended a layer coefficient value a_2 that ranges between 0.07 and 0.10 for the stone recharge bed. The value of 0.10 was chosen for the tool because it is the most frequently used.

***Step 5*:** Determine the composite subgrade resilient modulus (Schwartz & Hall, 2018).

- For stone recharge bed thicknesses greater than 12 in., it is recommended to use the following method called "Computing the Composite Subgrade Resilient Modulus":
  - Determine the analysis parameters:

    Find the values for: the modulus of the stone recharge bed or base/subbase (E_{base} or E_1), the existing subgrade resilient modulus (M_R), the thickness of the base (D_2 or h_1), the applied load (q), and the load diameter (a). Note: it is recommended to use an applied load q = 100 psi and a load diameter a = 6 in. for the analysis.
  - Determine the deflection for the two-layer system as expressed in Equation 39:

$$w_0 = \frac{1.5 * q * a}{E2} \times F_2 \tag{39}$$

    where, w_0 = deflection under load (in.), q = applied load (psi), a = diameter of circular load (in.), and F_2 = Burmister's Deflection Factor.

    F_2 is found from Figure 10 by extending a vertical line to a point corresponding to the ratio of stone recharge bed modulus to subgrade modulus (E_1/E_2) and extending a horizontal line to the *y*-axis to determine the deflection factor F_2. In order to avoid going through this process by hand in the COTPP, the values of F_2 were obtained from the graph by hand as a function of E_1/E_2 or E_{base}/M_R (base resilient modulus in psi / subgrade resilient modulus in psi) and h_1/a (stone recharge bed thickness in inches / load diameter in inches). The values h_1/a ranged from 0 to 6 at 0.1 increment and the values of E_1/E_2 ranged from 2 to 10,000 as shown in Figure 10. All the values obtained by hand were inserted in the COTPP as a database. The database can be found in Appendix A. In the tool, using the "HLOOKUP" function, F_2 values are pulled out easily from the database depending on the values of E_1/E_2 and h_1/a.

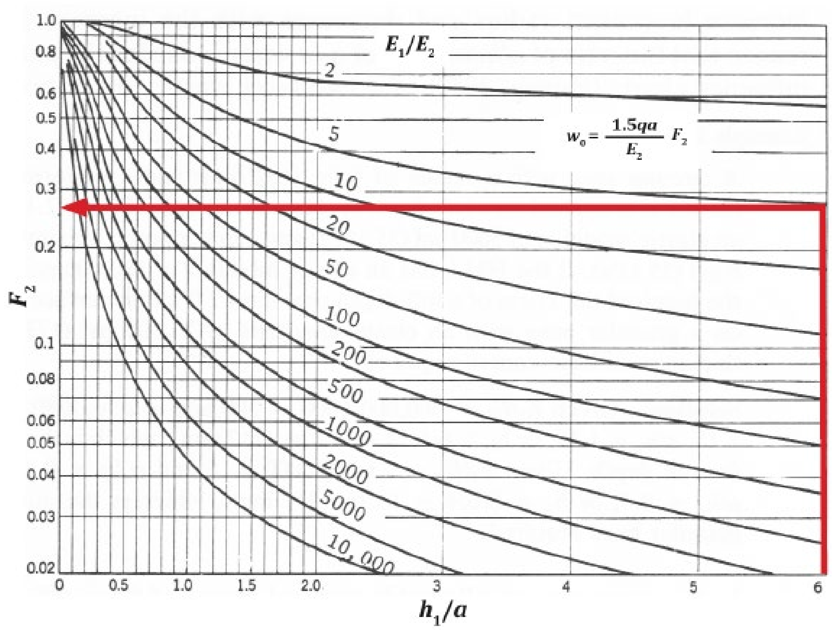


Figure 10. Burmister's deflection factor F2 graph (Schwartz & Hall, 2018)

- Determine the composite subgrade resilient modulus (E) of an equivalent single-layer composite system expressed as Equation 40.

$$E = \frac{1.5 \times q \times a}{w_0} \tag{40}$$

where, E = composite subgrade resilient modulus (psi), w_0 = deflection under load (in.), q = applied load (psi), and a = diameter of circular load (in.).

- For stone recharge bed thicknesses of 12 in. or less, NAPA stresses that the porous pavement can alternatively be designed as a conventional flexible pavement.

The more conservative resulting PA layer thickness of the two methods (composite subgrade vs. conventional 1993 method) should be selected. Since the composite subgrade method assumes that there are two definite layers, a geosynthetic fabric is recommended by NAPA between the base and the subgrade to achieve the separation in the field.

***Step 6*:** Derive the structural number from Equation 41 (Schwartz & Hall, 2018).

$$logW_{18} = Z_R S_0 + 9.36\log(SN+1) - 0.20 + \frac{log\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.4+\frac{1094}{(SN+1)^{5.19}}} + 2.32logM_R - 8.07 \qquad (41)$$

Equation 41 is from the AASHTO 1993 guide. All inputs are known except for the structural number (SN), which is obtained through back calculation in the tool.

***Step 7*:** Calculate the thickness of PA layer ($d_{asphalt}$) using Equation 42 (Schwartz & Hall, 2018).

$$d_{asphalt} = \frac{SN}{a_1} \qquad (42)$$

where, a_1 = layer coefficient of porous asphalt layer – 0.40 is recommended.

3.2.3 Permeable Interlocking Concrete Pavers (PICP)

According to the ICPI (Smith, 2012), the structural design of PICP should follow the AASHTO 1993 method. Therefore, based on the expected design ESALs, the properties of the soil or subgrade, and other structural properties, a required structural number is obtained and used to determine the unknown layer thicknesses (Weiss et al., 2017).

***Step 1*:** Estimate design traffic (W18).

Values of ESALs are typically relatively low because PICP is used mostly for parking lots and residential streets. The same process from step 1 for PA can also be used to obtain the design traffic for PICP.

***Step 2*:** Estimate reliability (R or Z_R), standard deviation (S_0), and allowable change in PSI (Smith, 2012).

- For Reliability (R or Z_R): R is usually 80%, which means that Z_R = -0.842
- For Standard Deviation (S_0): The recommended value is 0.45.
- For Allowable Change in PSI: $\Delta PSI = P_0 - P_t$

  P_0 is assumed to be 4.2 and P_t for low traffic can be taken as 2.5, which makes ΔPSI = 1.7.

***Step 3*:** Calculate subgrade resilient modulus (M_R) in psi using Equation 43 (Smith, 2012).

$$M_R\ (psi) = 2555 \times CBR^{0.64} \tag{43}$$

where, *CBR* = California Bearing Ratio – minimum 4% (Weiss et al., 2017).

***Step 4*:** Estimate the AASHTO layer coefficients (a_i) (Weiss et al., 2017; Smith, 2012).

- In PICP, the thickness of the surface course (pavers) is usually 3.125 in. and the underlying bedding layer of ASTM No. 8, 9 or 89 stone is 2 in. thick. According to ICPI, the layer coefficient value for pavers and the bedding layer in PICP (a_1) is estimated to range from 0.20 to 0.40 with a recommended average value of 0.30.
- For base layer of ASTM No. 57 stone, a layer coefficient (a_2) of 0.09 is assumed.
- For subbase layer of ASTM No. 2 stone, the assumed value (a_3) is 0.06.

***Step 5*:** Determine the structural number (SN).

The structural number is obtained from the AASHTO 1993 and expressed as Equation 44.

$$logW_{18} = Z_R S_0 + 9.36\,log(SN+1) - 0.20 + \frac{log\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.4+\frac{1094}{(SN+1)^{5.19}}} + 2.32logM_R - 8.07 \tag{44}$$

***Step 6*:** Calculate the thickness of subbase ASTM No. 2 layer ($d_{subbase}$) using Equation 34 (Schwartz & Hall, 2018). Since the thicknesses for PICP (3.125 in.), bedding layer and base ASTM No. 57 (4 in.) are fixed values, the only unknown is the thickness of the subbase ASTM No. 2 ($d_{subbase}$):

$$d_{subbase} = \frac{SN-\left(a_1*\left(d_{PICP}+d_{Bedding}\right)\right)-\left(a_2*d_{Base}*m_2\right)}{a_3*m_3} \tag{45}$$

where, d_{PICP} = depth of PICP (in.) – typically 3.125 in., $d_{Bedding}$ = depth of bedding layer (in.) – typically 2 in., a_1 = layer coefficient of PICP and bedding layer – typically 0.3, d_{Base} = depth of base layer ASTM No. 57 – typically 4 in., a_2 = layer coefficient of base layer – typically 0.09, a_3 = layer coefficient of subbase layer – typically 0.06, m_2 = m_3 = drainage coefficient – typically 1.0 for PP. This process is different from the NAPA guide process because the structural design is performed before the hydrological design for PICP. Therefore, the thickness of the subbase is first obtained then compared to the base/subbase layer found from the hydrological design to select the higher value.

3.3 Cost Estimation

PC and PA are constructed with similar techniques as for conventional pavements whereas the installation of permeable paving blocks may necessitate some hand placement, so additional labor costs might be needed. Although PPs and conventional pavements are composed of nearly the same required materials, the addition of geotextile material and the larger depth of the aggregate base/subbase represent the main differences between both practices. All the components of any type of PP have different initial construction costs that were provided by Wood & Volkert (2019) and Rehan et al. (2018) and summarized in Table 6. According to Wood & Volkert (2019), sweeping is the most appropriate maintenance technique and costs \$500 / year for a 0.75-acre parking lot. However, according to Rehan et al. (2018), the most effective maintenance technique is pressure washing and vacuuming which costs \$300 / year for a one-acre parking lot. In addition, the initial construction costs of all components of conventional pavements were provided by Wood & Volkert (2019), Rehan et al. (2018), and Walsh & Smallridge (2001) and are shown in Table 7. Concerning the GIs (bioretention and infiltration trench) included in the tool, their costs were provided by the US Environmental Protection Agency (US EPA, 2016) and shown in Table 7. It is important to note that the units of some construction costs were converted to proper units before their inclusion in the tool just as reported in Tables 6 and 7.

In the COTPP, only the construction costs were included for cost estimation calculations because maintenance is done yearly and is the same for all types of PP. A life cost analysis was performed by Rehan et al. (2018) to determine the most economical option between PA, PC, HMA, and PCC. It was concluded that PPs are more economical than conventional pavements in terms of maintenance costs. Therefore, PPs can become more economical than conventional pavements in the long term especially if stormwater treatment (drainage system) costs are added.

Table 6. Initial construction costs for permeable pavements materials

Items	Unit	Unit Cost
Excavation[1]	ft^3	\$0.56
Base course[2]	ft^3	\$1.56
Geotextile separation fabric[1]	ft^2	\$0.50
Pervious concrete[2]	ft^3	\$7.78
Porous Asphalt[2]	ft^3	\$5.33
Permeable interlocking concrete pavers[1]	ft^2	\$5

[1]Costs from Wood & Volkert (2019) (2019 dollars)
[2]Costs from Rehan et al. (2018) (2018 dollars)

Table 7. Initial construction costs of materials for conventional pavements and other GIs

Items	Unit	Unit Cost
Excavation[1]	ft^3	\$0.56
Base course at 6”[2]	ft^2	\$0.89
Hot-Mix Asphalt (HMA) at 3.75”[2]	ft^2	\$2.22
Portland cement concrete (PCC) at 6”[2]	ft^2	\$3.33
Interlocking concrete pavers (ICP) at 4”[3]	ft^2	\$3.02
Bioretention[4]	ft^3	\$15.46
Infiltration trench[4]	ft^3	\$12.49

[1]Costs in 2019 dollars from Wood & Volkert (2019)
[2]Costs in 2018 dollars from Rehan et al. (2018)
[3]Costs in 2001 dollars from Walsh & Smallridge (2001)
[4]Costs in 2016 dollars from US EPA (2016)

The costs for each type of permeable pavements (PP), conventional pavements (CP), and other GIs are estimated in the COTPP using Equation 46, Equation 47, and Equation 48, respectively.

$$\boldsymbol{PP\ cost}\ (\$) = \left(PP\ unit\ cost\ \left(\frac{\$}{ft^3}\right) \times PP\ surface\ layer\ volume\ \left(ft^3\right)\right) + \left(\frac{Base}{Subbase} course\ unit\ cost\ \left(\frac{\$}{ft^3}\right) \times \frac{Base}{Subbase} volume\left(ft^3\right)\right) + \left(Geotextile\ fabric\ unit\ cost\ \left(\frac{\$}{ft^2}\right) \times PP\ area\ \left(ft^2\right)\right) + \left(Excavation\ unit\ cost\ \left(\frac{\$}{ft^3}\right) \times total\ PP\ volume\ (ft^3)\right) \quad (46)$$

$$\boldsymbol{CP\ cost}\ (\$) = \left(CP\ unit\ cost\ \left(\frac{\$}{ft^2}\right) \times CP\ area\ \left(ft^2\right)\right) + \left(Base\ course\ unit\ cost\ \left(\frac{\$}{ft^2}\right) \times Base\ course\ area\ \left(ft^2\right)\right) \quad (47)$$

$$\boldsymbol{Other\ GIs\ cost}\ (\$) = (GI\ unit\ cost\left(\frac{\$}{ft^3}\right) \times GI\ storage\ volume\ (ft^3)) + (Excavation\ unit\ cost\left(\frac{\$}{ft^3}\right) \times total\ GI\ volume\ (ft^3)) \quad (48)$$

Since the costs shown in Tables 6 and 7 above are in dollars of previous years, they need to be adjusted to dollars of the current year. This adjustment can be done for inflation using Equation 49 with the 20-city average value of Engineering News Record (ENR) Construction Cost Index (CCI) and for location to account for regional differences in construction costs (i.e., materials and labor) using Equation 50 with the most recent regional factors. The ENR CCI values for recent years can be found on the Engineering News Record website (ENR, 2021). For a more efficient optimization process, the user is required to obtain the most recent ENR values and adjust the costs from Tables 6 and 7.

$$Cost\ (present) = Cost\ (base\ year) \times \frac{ENRCCI\ (present)}{ENRCCI\ (base\ year)} \tag{49}$$

$$Cost\ (regional) = Cost\ (national) \times \frac{ENRCCI\ (regional)}{ENRCCI\ (national)} \tag{50}$$

3.4 Cost Optimization Algorithm

To optimize the costs for PPs, a method was developed for the COTPP called the "Combination of Green Infrastructures or GIs". This method is similar to a heuristic search and consists of combining PPs with other green infrastructure, with conventional pavements, or all together to select the most cost-effective option. To achieve that goal, three options for combination were created for the tool and are summarized in Table 8.

Table 8. Combination options for cost optimization of permeable pavements

Options	**Components**		
	Permeable pavements (PC, PA, or PICP)	Conventional pavements (HMA, PCC, or ICP)	Other GIs (Infiltration trench or Bioretention)
A	✓	✓	
B	✓		✓
C	✓	✓	✓

Since PPs are usually more expensive to build on their own compared to conventional pavements, combining them with cheaper practices such as conventional pavements and other green infrastructure can reduce costs and help save money. Therefore, an algorithm was created in the COTPP for each combination option shown in Table 8. The algorithm allows the user to input certain boundaries based on known parameters of the project site considered and the components being used for combination. Once the inputs are entered, the dimensions of each component are calculated in the "Optimization" tab of the tool as well as the construction costs, which are added to determine the total cost of the combination option. Next, the final construction costs of all options are displayed, and the user can select the option that is more cost effective compared to PPs alone. The calculation process of the algorithm is further explained below for each option.

Option A: Combination of permeable pavement and conventional pavement.

With this option, the area of the PP is reduced because the materials of its surface layer are the most expensive. That area is replaced by conventional pavement and the depth of the reservoir layer is increased to keep the storage capacity required for stormwater runoff. To achieve that, the following steps are followed in the tool:

1. In the "User Interface" Tab, the types of permeable pavement and conventional pavement are selected by the user from the dropdown list.
2. Then, the tool requires the user to type in the range of conventional pavement surface area desired in terms of percentage as a function of the permeable surface area.

   e.g.: A user was originally planning to use an area of 16000 ft^2 to build a 100% pervious pavement parking lot. They decide to use the tool for cost optimization and desires to use between 10 and 50% of the initial pervious area of 16000 ft^2 as conventional pavement.

3. In the “Optimization” Tab, the “randbetween” function is used to generate 10,000 random numbers within the range selected in Step 2. Those numbers are converted to ft^2 and are added to any contributing impervious areas (such as roofs, hardscapes etc.) to obtain new impervious areas for the design.

   e.g.: 10% and 50% of 16000 ft^2 are 1600 ft^2 and 8000 ft^2, respectively, and 10,000 random values between those two numbers are selected.

4. New pervious areas are calculated by subtracting the new impervious areas found in step 3 from the overall initial permeable area.

   e.g.: every random number between 1600 ft^2 and 8000 ft^2 is subtracted from 16000 ft^2.

5. Using the new areas from Steps 3 and 4, a new base/subbase for the permeable pavement is designed following the hydrological design method precedingly discussed in Chapter 3.

6. All construction costs for permeable pavements and conventional pavements are used to calculate the overall cost for Option A combination. 10,000 trials are performed and the trial with the lowest overall cost is selected for Option A and displayed in the “User Interface” Tab.

   Option B: Combination of permeable pavement and other GI (infiltration trench or bioretention).

The area of the permeable pavement stays the same in this combination while the depth of the reservoir layer is decreased thanks to the addition of an infiltration trench or bioretention, which helps store some or most of the runoff. The following steps are followed in the tool:

1. In the “User Interface” Tab, the user is asked to enter the maximum available area for the other GIs (Infiltration trench or Bioretention) to occupy on the site.

   e.g.: Using the same example from Option A, the user can decide that only 1000 ft^2 of the

16000 ft^2 area are available to install an infiltration trench or bioretention.

2. The user is asked to type in the range of area for "other GIs" desired in terms of percentage as a function of the maximum available area from Step 1.

   e.g.: User can decide to only use between 50 and 90% of the available area (1000 ft^2) to install an infiltration trench or bioretention on site.

3. The user is also asked to enter a range of stormwater storage capacity for other GIs in terms of percentage as a function of the overall treatment volume T_v calculated based on the design precipitation.

   e.g.: If the precipitation is 1.2 in. and the treatment volume is 4560 ft^3, the user can decide to store between 20% to 80% of 4560 ft^3 in an infiltration trench or bioretention and the remaining will be stored in the reservoir layer of the permeable pavement.

4. In the "Optimization" Tab, the "randbetween" function is used to generate random numbers for new GI treatment volume within the range selected in Step 3.

   e.g.: 1500 random values between 20% and 80% of 4560 ft^3 are generated.

5. New GI storage volumes are obtained by dividing the new GI treatment volumes from step 4 by 0.4 because aggregates in the media of bioretention and infiltration trench have a void ratio of 40%. It is important to note that the COTPP provides only the storage volume (media) of bioretention or infiltration trench. For more information on detailed design of bioretention or infiltration trench, see the City of Birmingham (2019) manual.

6. The "randbetween" function is used to generate 10,000 random numbers for new GI surface area within the range selected in Step 2.

7. New GI depths are obtained by dividing the new GI storage volumes from Step 5 by new GI surface area from Step 6.

8. New treatment volumes for permeable pavement are obtained by subtracting the new GI treatment volumes (Step 4) from the initial overall treatment volume (i.e.: $T_v = 4560ft^3$).
9. Using results from previous steps, new base/subbase thicknesses for the permeable pavement are calculated by: (1) dividing the new treatment volume from Step 8 by 0.40 to account for 40% void ratio and obtain total PP volume. (2) then dividing total PP volume by pervious area to obtain the base/subbase thickness.
10. All construction costs for permeable pavements and other GIs are used to calculate the overall cost for Option B combination. 10,000 trials are performed and the trial with the lowest overall cost is selected for Option B and displayed in the "User Interface" Tab.

Option C: Combination of permeable pavement, conventional pavement, and other GI.

1. Since Option C = Option A + Option B, the new total PP volumes for permeable pavement (from Step 9 in Option B) and the new pervious areas (from Step 4 in Option A) are used to design new base/subbase thicknesses.
2. All construction costs for permeable pavements, conventional pavements, and other GIs are used to calculate the overall cost for Option C combination. 10,000 trials are performed and the trial with the lowest overall cost is selected for Option C and displayed in the "User Interface" Tab.

For the design of the three Options, the minimum base/subbase thickness of each PP type can be entered in the "Detailed Inputs" tab of the tool. This will be accounted for in the design of PPs. When adjusting Options' inputs in the tool for optimization, the user should know and consider the maximum base/subbase thickness that they plan to allow, the depth of the ground water table, and all other design constraints previously stated in Chapter 2. This will help obtain acceptable PP depths and GIs' depths.

Another option (Option D) could have been added to combine conventional pavements with other GIs, but Ellis (2020) already developed a tool that is able to execute this type of combination analysis and provide acceptable results. The practical tool developed by Ellis will be combined in the near future with the COTPP from this study to obtain one definitive optimization tool. In Ellis' practical tool, there is a possibility to combine all types of land (not just impervious parking lot) with structural GIs such as bioretention and infiltration trench. Furthermore, Ellis included in his tool a hydrologic model that generate pre-development and post-development runoff hydrographs for a flood protection design storm and calculate the required storage volume for a detention basin. Thus, results such as treatment volume (T_v) to be treated by either bioretention and infiltration trench from Options B and C of this tool can be transferred to Ellis' tool if excess runoff is a concern. Ellis' tool can be used to calculate the required storage volume for a detention basin to attenuate peak flows and capture excess runoff from the structural GIs.

CHAPTER 4: METHODS FOR SENSITIVITY ANALYSIS AND CASE STUDY

4.1 Sensitivity Analysis

In this section, the results from the COTPP are compared to the results from design examples published in the different design guidelines previously discussed in Chapter 3 in order to validate the reliability and effectiveness of the tool.

4.1.1 Pervious Concrete

Two design scenarios were simulated in both the software called PerviousPave and the COTPP to compare their structural and hydrological results. The comparisons were done to ensure that every output from the COTPP is as valid as the results from existing tools like PerviousPave. The general inputs for the two design scenarios used in both tools are shown in Table 9:

Table 9. General inputs for PC sensitivity analysis scenarios

Design life	20 years
Design reliability	80%
Projected application	Category B - residential/parking lot for PerviousPave
Annual truck traffic growth (g)	2%
Minimum depth of the reservoir layer	6in.
Elastic modulus of base/subbase layer	15,000psi
28-day flexural strength of pervious concrete	400psi
Modulus of elasticity	2,700,000psi
Axle load edge support?	No
Pervious concrete area	16,000ft^2
Non-pervious area	32,000ft^2
Percent voids of reservoir layer material	40%

Scenario 1: This scenario had for purpose to determine the thickness of the PC surface layer at different traffic levels and soil's strengths. For that, the Average Daily Truck Traffic, one-way (ADTT) varied from 2 to 10 (increment of 1) and the CBR of the soil varied from 4% to 15% (increment of 1). Different CBR means different subgrade resilient modulus (M_r) as well as different composite modulus of subgrade/subbase reaction (k).

Scenario 2: The goal of this scenario is to determine the thickness of the reservoir layer or base/subbase layer based on varying soil infiltration rates and design storm precipitations. To achieve that goal, the infiltration rate of the soil was increased from 0.5 in./hr to 2 in./hr (increment of 0.25) and the rainfall depth was increased from 1 in. to 6 in. (increment of 0.25).

4.1.2 Porous Asphalt

In Appendix B of the NAPA guide written by Schwartz & Hall (2018), tables were provided as design examples as well as references for porous asphalt designers. Those tables show the required PA surface layer thickness as a function of base/subbase thickness and subgrade modulus. The tables correspond to different design traffic (ESALs) that vary from 27,000 ESALs to 5 million ESALs. Only three design scenarios were reproduced in the COTPP for results comparisons. The chosen design ESALs were 27,000 ESALs, 110,000 ESALs and 820,000 ESALs because PPs are usually designed for low traffic areas. Since the hydrologic design is the same for all PP types and was previously evaluated in pervious concrete section, only the structural design of PA was analyzed. The design assumptions are shown in Table 10:

Table 10. General inputs for PA sensitivity analysis scenarios

Coefficients for porous asphalt layer (a_1)	0.40
Coefficients for base/subbase layer (a_2)	0.10
Modulus of the base/subbase	20,000psi
Design reliability	75%
Standard deviation (S_0)	0.45
Initial serviceability (p_i)	5
Terminal serviceability (p_t)	2.5
Allowable serviceability decrease (ΔPSI)	2.5
Circular load radius (a)	5.35in.
Load pressure (q)	100psi

Scenario 1: Traffic level = 27,000 ESALs; base/subbase thickness varies from 6 in. to 30 in.; design subgrade resilient modulus varies from 2,000 psi to 10,000 psi.

Scenario 2: Traffic level = 110,000 ESALs; base/subbase thickness varies from 6 in. to 30in.; design subgrade resilient modulus varies from 2,000 psi to 10,000 psi.

Scenario 3: Traffic level = 820,000 ESALs; base/subbase thickness varies from 6 in. to 30 in.; design subgrade resilient modulus varies from 2,000 psi to 10,000 psi.

4.1.3 Permeable Interlocking Concrete Pavers (PICP)

The Interlocking Concrete Pavement Institute (ICPI) provided a table of recommended minimum PICP base and subbase thicknesses in their industry guidelines. The thicknesses were obtained at different traffic levels (design ESALs) and CBR of the subgrade. The design traffic (ESALs) varies from 50,000 ESALs to 1 million ESALs and the CBR varies from 4 to 10. The same design scenario was simulated in the COTPP to compare to ICPI guide values and check the reliability of the tool's results. The design assumptions for the scenario are shown in Table 11:

Table 11. General inputs for PICP sensitivity analysis scenarios

Coefficient for the combination of PICP layer and the bedding layer (a_1)	0.3
Coefficient of the base layer (a_2)	0.09
Coefficient of the subbase layer (a_3)	0.06
Modulus of the base/subbase	15,000psi
Design reliability	80%
Standard deviation (S_0)	0.45
Initial serviceability (p_i)	4.2
Terminal serviceability (p_t)	2.5
Allowable serviceability decrease (ΔPSI)	1.7

4.2 Case Study

The purpose of the case study is to verify that the COTPP can in fact help optimize the design and costs of PPs. To achieve that, the construction costs of an existing permeable parking lot were input in the tool and the cost results obtained were compared to the actual final construction cost of that existing parking lot. Since the COTPP was developed for Alabama designers and engineers, the site selected for the case study (Figure 11) is a parking lot located in Lee county, Alabama. In addition, the COTPP optimizes costs by optimizing the PP design via the combination of GIs and/or conventional practices. Consequently, the selected existing site is composed of HMA pavement for entrance lane and driving lanes, PICP for parking spaces, and a bioretention beside the parking lot. During a rain event, the stormwater that falls on the PICP enters a stone recharge bed and infiltrates the underlying soil and/or is directed to the bioretention through an underdrain pipe. Runoff from the impervious HMA pavement is directed through an outflow to the bioretention system. Water that enters the bioretention is treated and infiltrates into the native soil or leaves the site through an outflow to an existing stormwater conveyance network. Thus, it can be said that the site serves as both a parking lot and a stormwater treatment system. The dimensions of each component of the site were obtained from provided AutoCAD drawings and used to redraw cross sections shown in Figures 12 and 13 for the case study. According to the NRCS web soil survey, the general hydrologic soil group of soils at the site location is B (NRCS,1986). During the construction planning, the property acquisition cost and the construction cost were estimated to be \$1.2 million and \$1.0 million, respectively. The bid prices for the construction of some components of the site were obtained and converted to appropriate unit costs for the case study through calculations shown in Appendix F. It was assumed that the costs were in 2019 dollars, which is the year when the parking lot was built. The costs were not adjusted for inflation before their inclusion in the tool because the results from the tool had to be compared to the final cost estimate in the bid documents. The summary of the unit costs is shown in Table 12 and the final construction cost for PICP, HMA, and bioretention at the selected site was \$305,918.

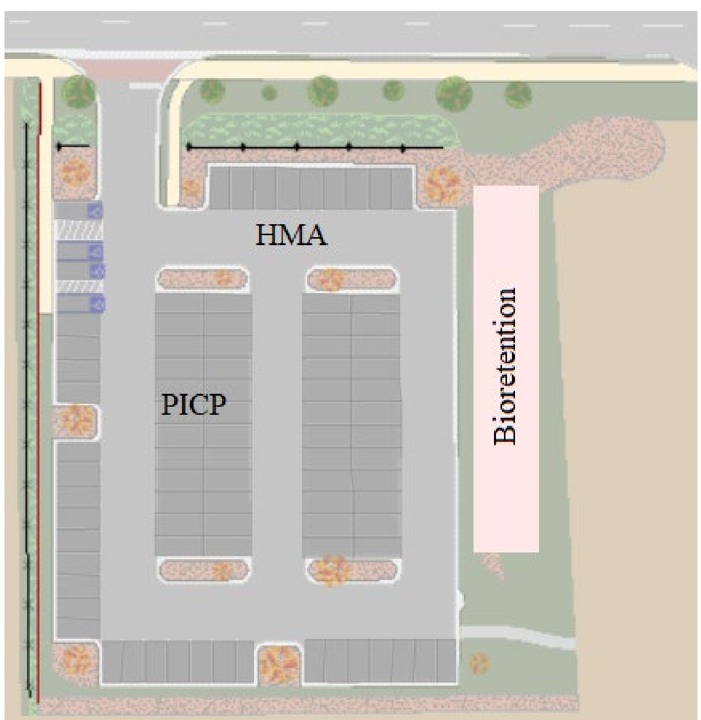


Figure 11. Case Study Site - PICP (7800 ft^2), HMA (29250 ft^2), and Bioretention (5544 ft^2)

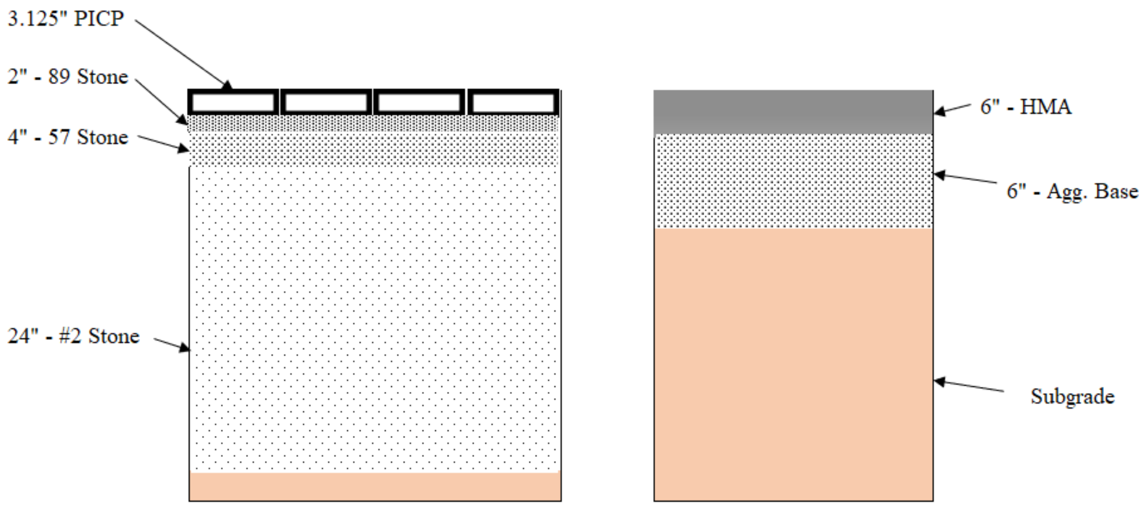


Figure 12. PICP and HMA cross-sections (case study site)

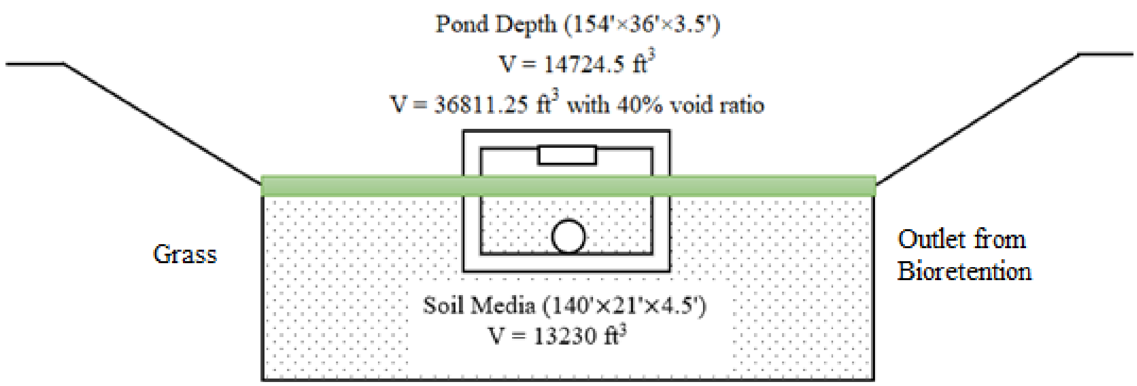


Figure 13. Bioretention cross-section (case study site)

Table 12. Construction unit costs for existing site components (from provided bid documents)

Components	Bioretention	PICP	HMA	Crushed Aggregate Base	Excavation
Unit costs	0.64 $/ft^3$	15.38 $/ft^2$	1.61 $/ft^2$	0.805 $/ft^2$ 1.61 $/ft^3$	0.73 $/ft^3$

Two cost optimization scenarios were simulated to determine if the COTPP is able optimize the original design of the existing site and produce lower final costs compared to the actual final cost ($305,918) of the site. The common general inputs for the two scenarios are as follows:

The design life was 20 years, the design reliability was 80 %, and the projected application was category B. The annual truck traffic growth (g) was 2%, the minimum depth of the reservoir layer was 9 in., and the elastic modulus of base/subbase layer was 15,000 psi. The entire parking lot area was 37050 ft^2, the non-pervious area was 0 ft^2, and the percent voids of reservoir layer material was 40%. The subgrade infiltration rate was left at the minimum of 0.5 in./hr and the HSG B was used for the soil. To obtain the exact same PICP cross section as the existing one, the design storm precipitation used in the tool was 11.7 in.

Scenario 1: In this scenario, the areas of PICP and HMA were varied for cost optimization.

Scenario 2: In this scenario, only the amount of treatment volume to be treated by the bioretention was varied while maintaining the same PICP and HMA areas.

CHAPTER 5: PRESENTATION OF RESULTS AND DISCUSSION

5.1 Cost Optimization Tool for Permeable Pavements (COTPP)

The product of this study is the practical tool called the "Cost Optimization Tool for Permeable Pavements (COTPP)". Chapter 3 explained the methodology used to create this tool. The COTPP is an Excel spreadsheet-based tool composed of six (6) tabs or worksheets. The worksheets are described and discussed in order as follows:

<u>*User Interface*</u>*:* This worksheet represents the main Tab where the user enters all necessary inputs for the design and receives the outputs. It is one of the two worksheets that allow the user to make changes. It contains four (4) parts that only display the most important parameters needed by the designer / engineer when using the tool. First, there is the PP inputs section as shown in Figure 14 where the user can enter general information of the project and inputs values (traffic, structural and hydrological properties) for the design of permeable pavements. The user should click on the "Run" button to display the results on the PP outputs section.

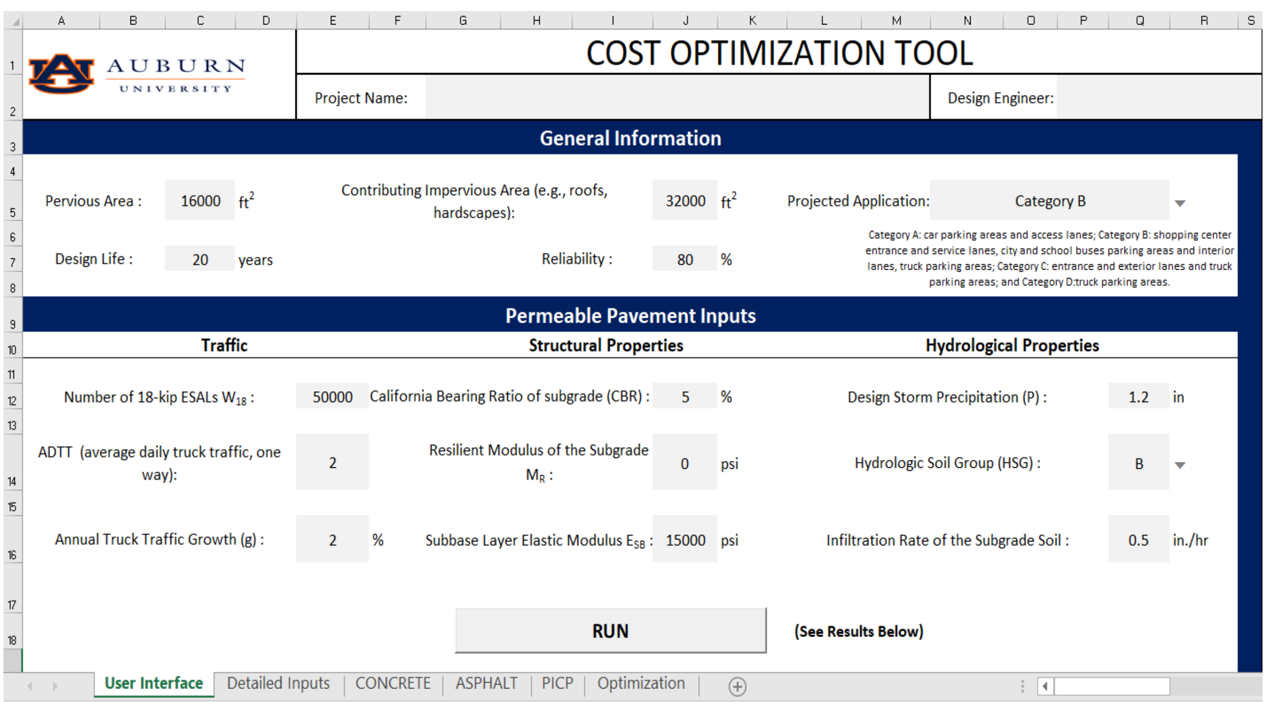


Figure 14. Permeable pavements (PP) inputs section in user interface

The PP outputs section is shown in Figure 15. In this section, the dimensions of the PP surface layer, bedding layer, and base/subbase layer, are displayed. The overall construction cost for each type of PP is shown and the least expensive is recommended.

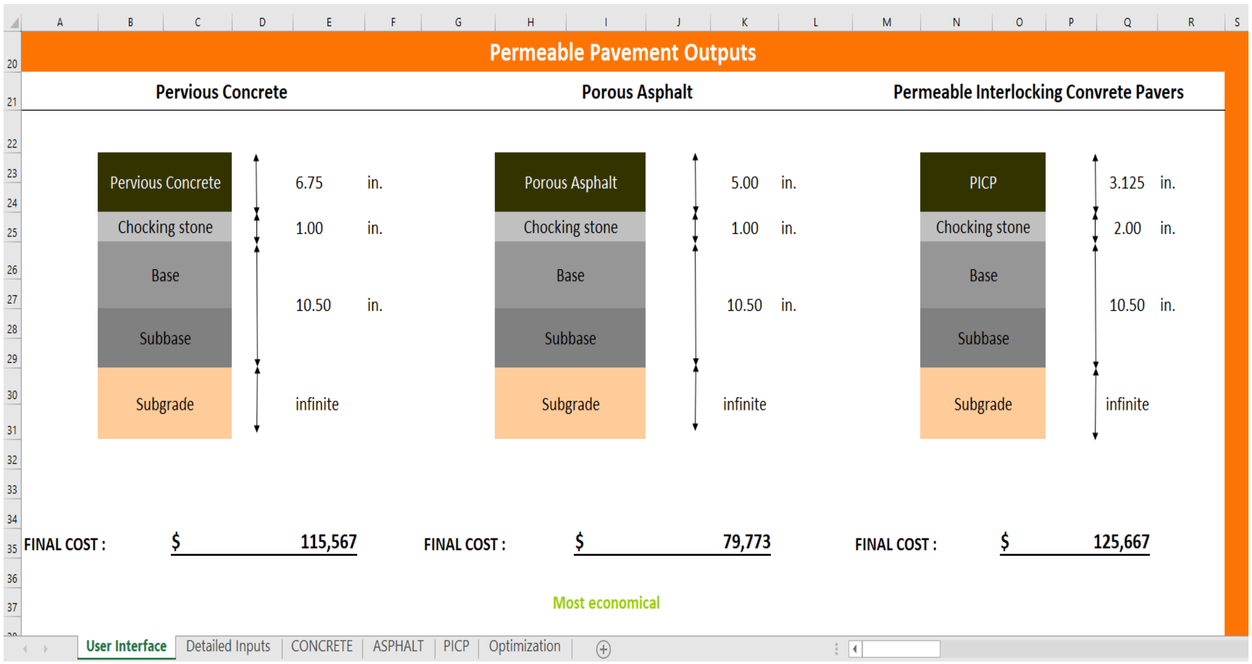


Figure 15. Permeable pavements outputs section in user interface

The third section in the user interface tab is the optimization inputs section and can be seen in Figure 16. This section allows the user to choose the types of permeable pavement, conventional pavement, and other GI that is suitable for the optimization analysis. Certain criteria or boundaries can be established by the user as explained in Chapter 3 for the optimization to occur. The user should click on the “Run” button to obtain the results in the optimization outputs section.

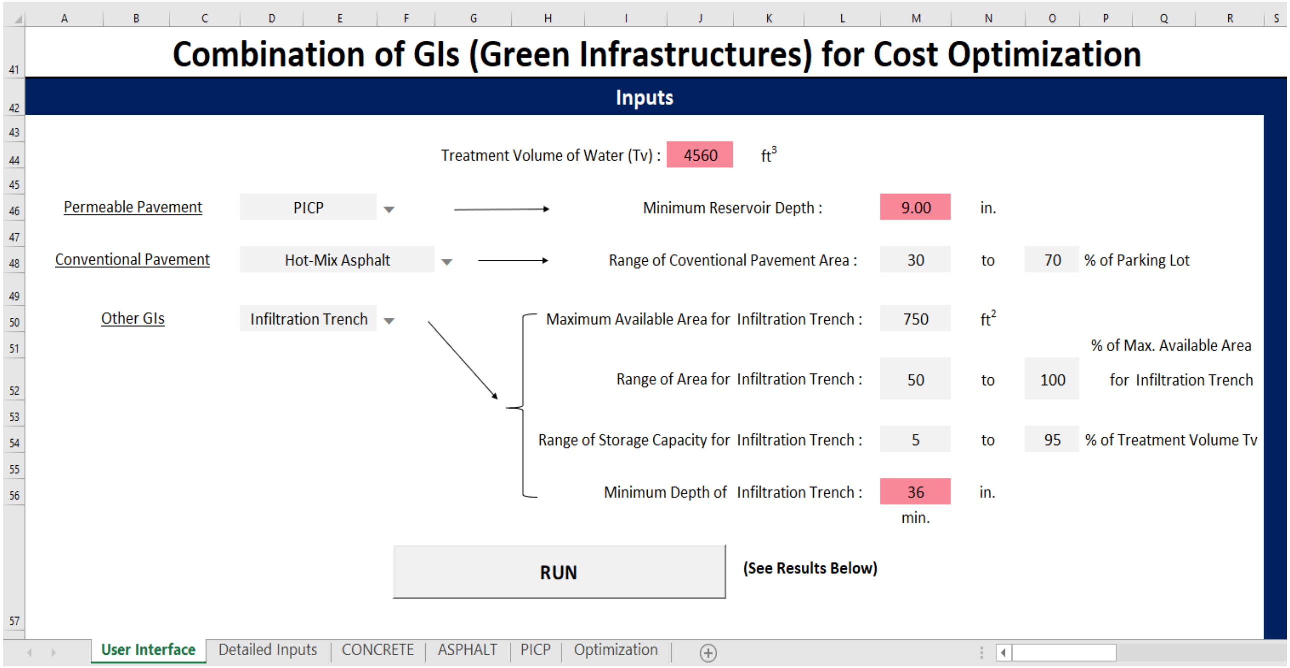


Figure 16. Optimization inputs section in user interface

The last section of the user interface worksheet is the optimization outputs section shown in Figure 17. In this section, the results of each combination option are displayed with the recommended dimensions of each component and the overall construction cost. The user can compare prices and choose the most economical option to use for the specific project. It is important to note that the design calculations for cost optimization are done using the inputs from the user interface tab and the next tab called “Detailed Inputs”. The users can enter their own input values, but if not the inputs values stated in Chapter 3 will be used as default values.

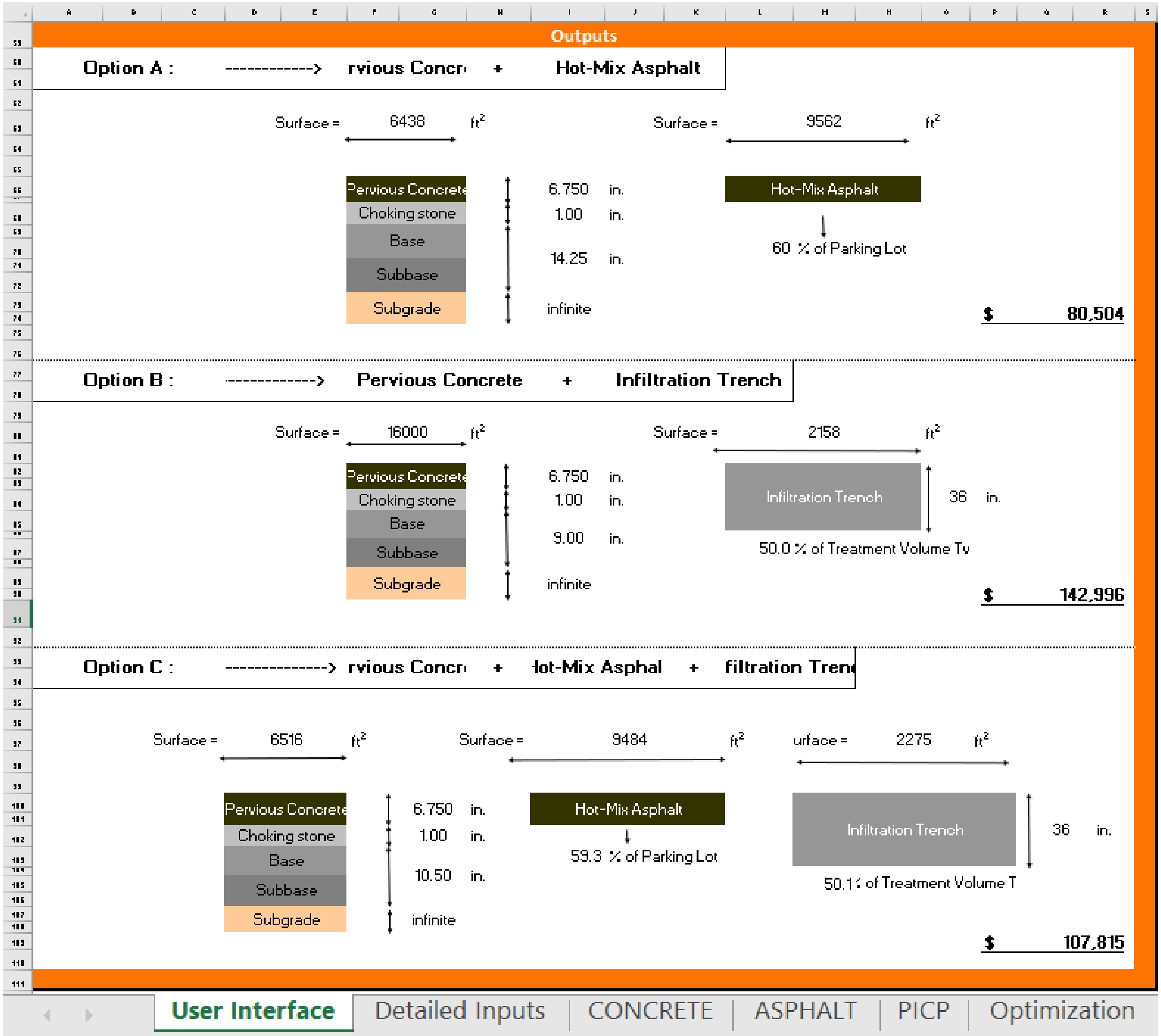


Figure 17. Optimization outputs section in user interface

Detailed Inputs: This worksheet represents the location where all other secondary inputs that are not shown in the user interface tab can be entered by the user. It is the last of the two worksheets that allow the user to make changes in the tool. It contains two (2) parts that are needed for the design of PP and construction cost calculations. The first part is the PP inputs section as shown in Figure 18 where the user can enter structural and hydrological inputs for each type of PP.

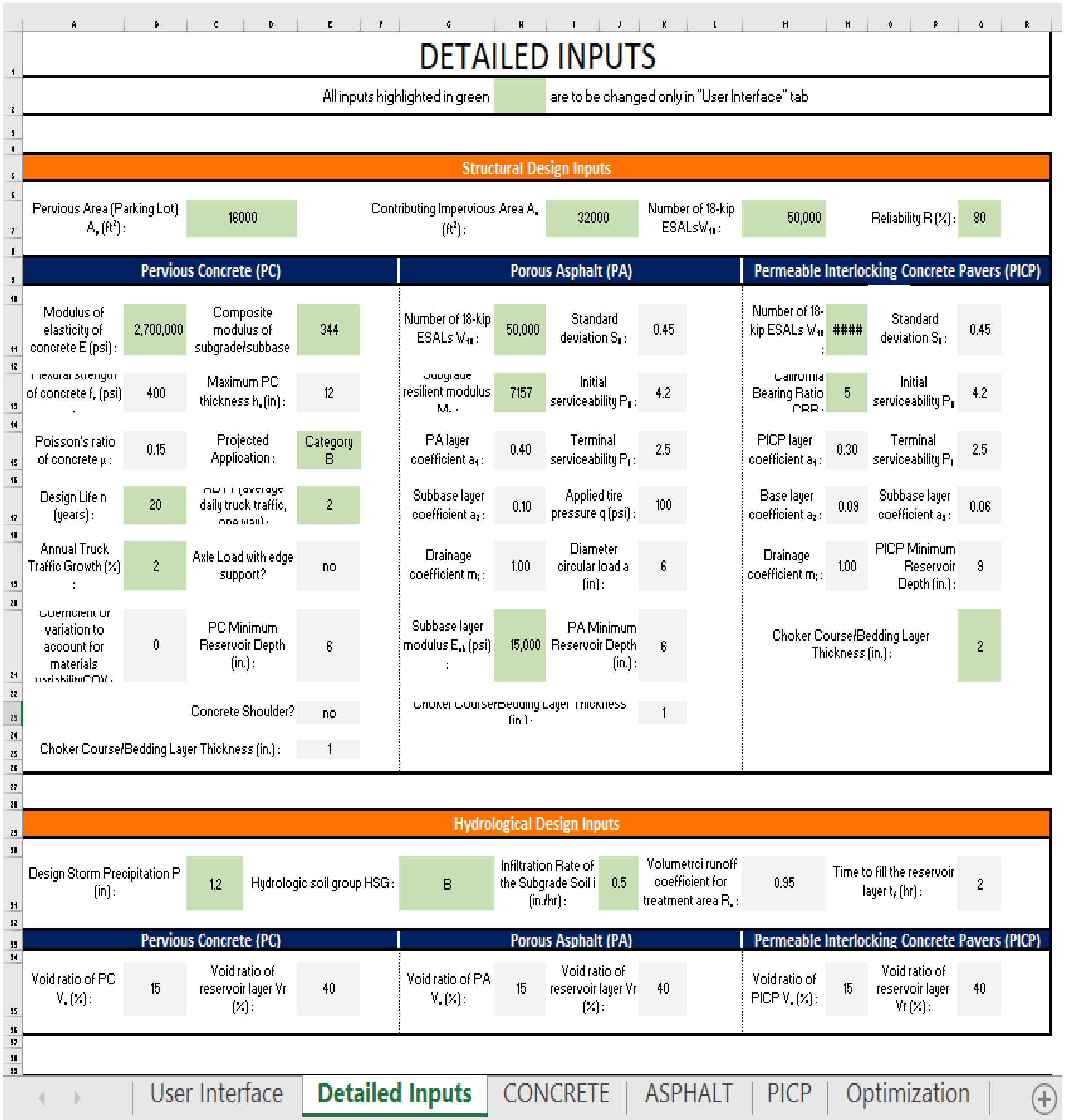


Figure 18. Permeable pavement (PP) inputs section in detailed inputs tab

The second part is the construction cost section as shown in Figure 19 where all cost data are stored to be used in the cost optimization process. The costs for all types of PP, conventional pavements, other GIs, and conventional drainage systems are shown.

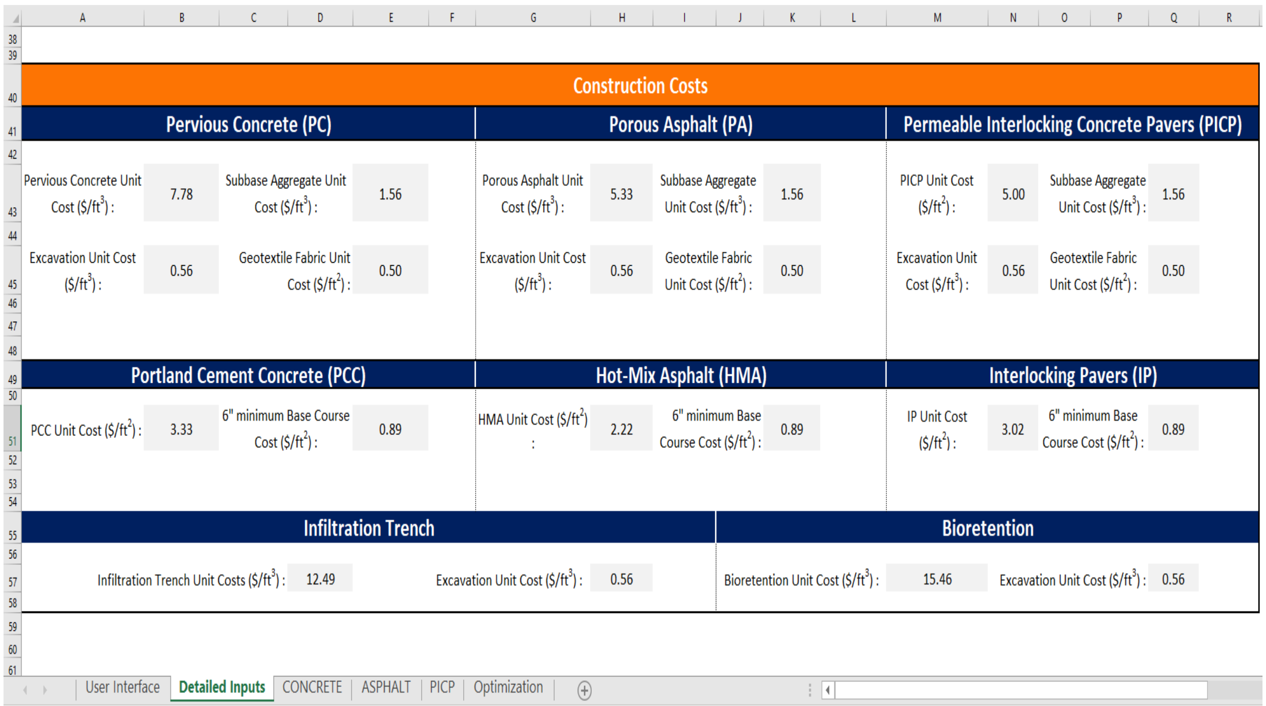


Figure 19. Construction cost section in detailed inputs tab

Concrete: The structural and hydrological designs of PC are performed in this worksheet (Figure 20) using the inputs from the user interface tab and the detailed inputs tab. The design process followed for calculations was discussed in Chapter 3. This worksheet is locked and can only be unlocked with a password provided to the principal owner of the tool.

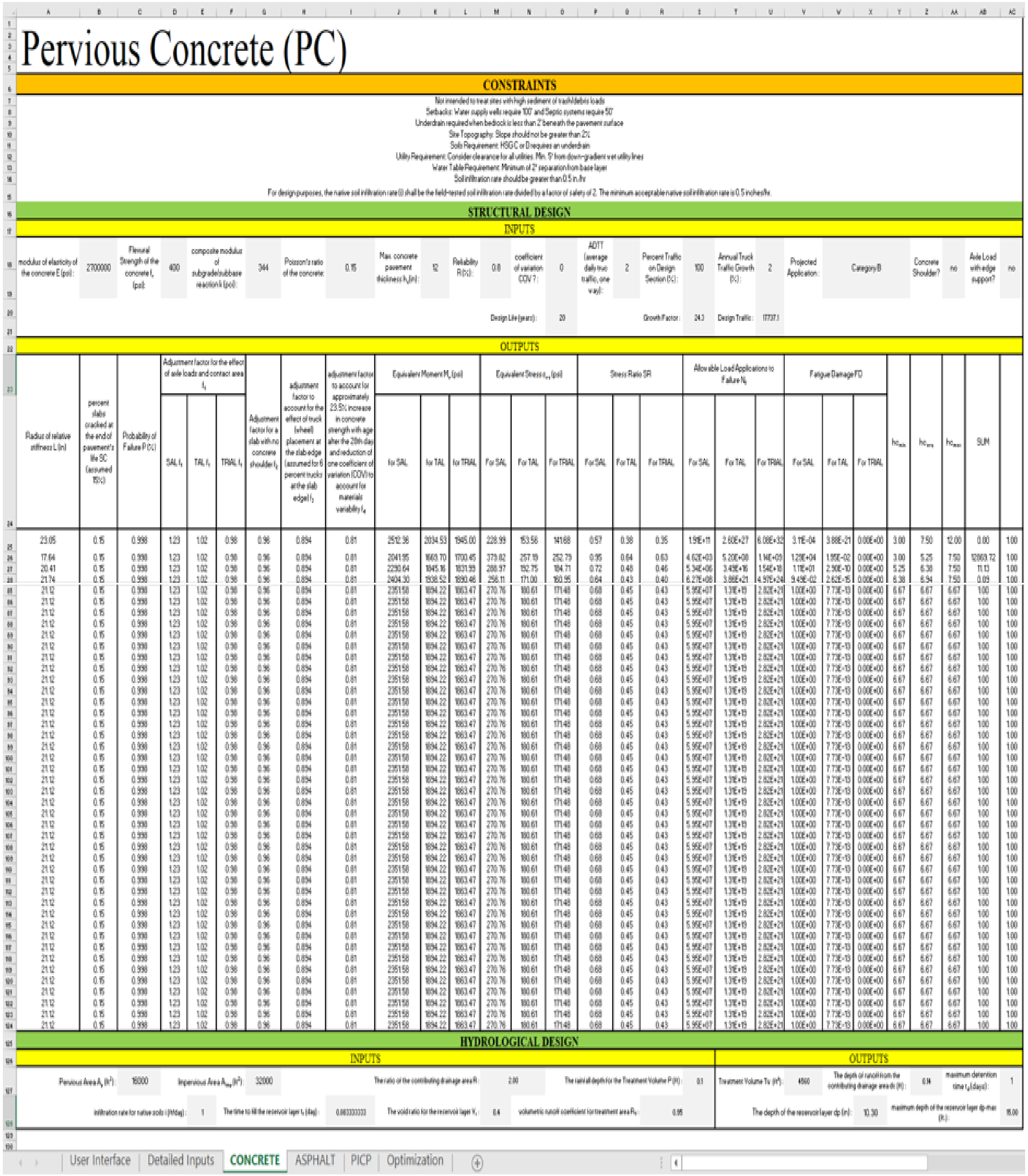
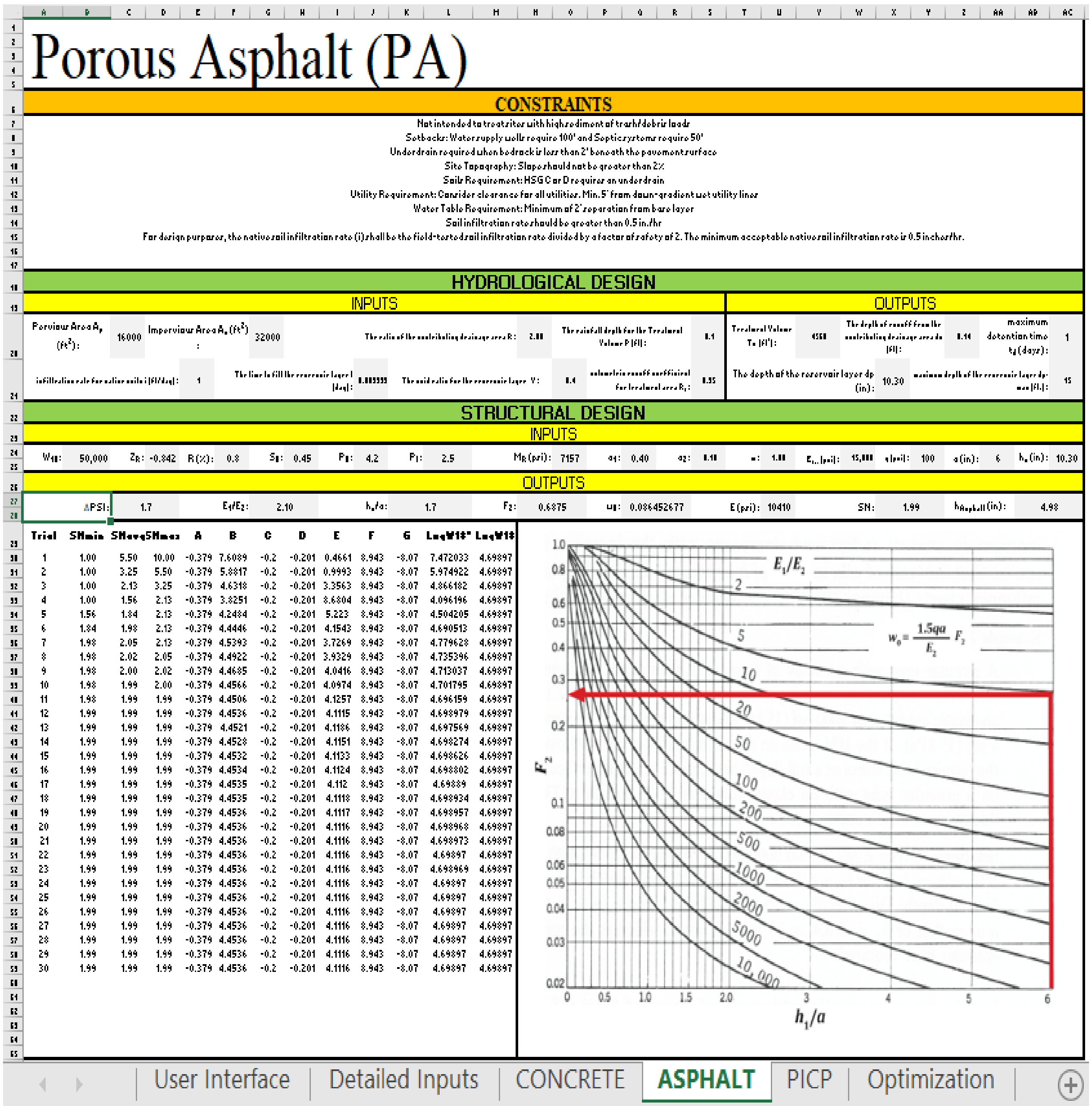


Figure 20. Pervious Concrete tab

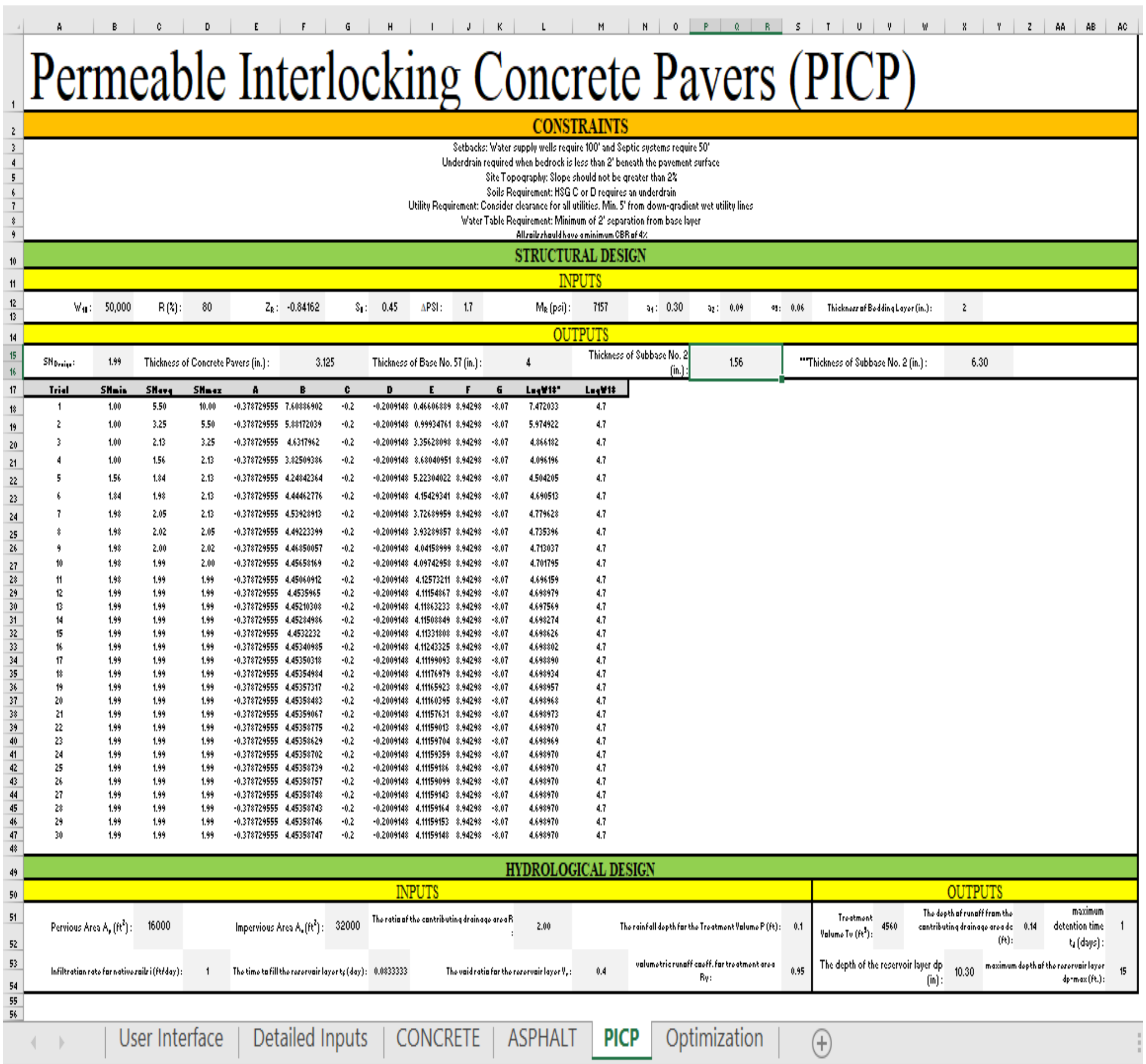
Asphalt: The structural and hydrological designs of PA are performed in this worksheet (Figure 21) using the inputs from the user interface tab and the detailed inputs tab. The design process followed for calculations was discussed in Chapter 3. This worksheet is locked and can only be unlocked with a password provided to the principal owner of the tool.



Trial	SNmin	SNavg	SNmax	A	B	C	D	E	F	G	LogW18*	LogW18
1	1.00	5.50	10.00	-0.379	7.6089	-0.2	-0.201	0.4661	8.943	-8.07	7.472033	4.69897
2	1.00	3.25	5.50	-0.379	5.8817	-0.2	-0.201	0.9993	8.943	-8.07	5.974922	4.69897
3	1.00	2.13	3.25	-0.379	4.6318	-0.2	-0.201	3.3563	8.943	-8.07	4.866182	4.69897
4	1.00	1.56	2.13	-0.379	3.8251	-0.2	-0.201	8.6804	8.943	-8.07	4.096196	4.69897
5	1.56	1.84	2.13	-0.379	4.2484	-0.2	-0.201	5.223	8.943	-8.07	4.504205	4.69897
6	1.84	1.98	2.13	-0.379	4.4446	-0.2	-0.201	4.1543	8.943	-8.07	4.690513	4.69897
7	1.98	2.05	2.13	-0.379	4.5393	-0.2	-0.201	3.7269	8.943	-8.07	4.779628	4.69897
8	1.98	2.02	2.05	-0.379	4.4922	-0.2	-0.201	3.9329	8.943	-8.07	4.735396	4.69897
9	1.98	2.00	2.02	-0.379	4.4685	-0.2	-0.201	4.0416	8.943	-8.07	4.713037	4.69897
10	1.98	1.99	2.00	-0.379	4.4566	-0.2	-0.201	4.0974	8.943	-8.07	4.701795	4.69897
11	1.98	1.99	1.99	-0.379	4.4506	-0.2	-0.201	4.1257	8.943	-8.07	4.696159	4.69897
12	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1115	8.943	-8.07	4.698979	4.69897
13	1.99	1.99	1.99	-0.379	4.4521	-0.2	-0.201	4.1186	8.943	-8.07	4.697569	4.69897
14	1.99	1.99	1.99	-0.379	4.4528	-0.2	-0.201	4.1151	8.943	-8.07	4.698274	4.69897
15	1.99	1.99	1.99	-0.379	4.4532	-0.2	-0.201	4.1133	8.943	-8.07	4.698626	4.69897
16	1.99	1.99	1.99	-0.379	4.4534	-0.2	-0.201	4.1124	8.943	-8.07	4.698802	4.69897
17	1.99	1.99	1.99	-0.379	4.4535	-0.2	-0.201	4.112	8.943	-8.07	4.69889	4.69897
18	1.99	1.99	1.99	-0.379	4.4535	-0.2	-0.201	4.1118	8.943	-8.07	4.698934	4.69897
19	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1117	8.943	-8.07	4.698957	4.69897
20	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1116	8.943	-8.07	4.698968	4.69897
21	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1116	8.943	-8.07	4.698973	4.69897
22	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1116	8.943	-8.07	4.69897	4.69897
23	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1116	8.943	-8.07	4.698969	4.69897
24	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1116	8.943	-8.07	4.69897	4.69897
25	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1116	8.943	-8.07	4.69897	4.69897
26	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1116	8.943	-8.07	4.69897	4.69897
27	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1116	8.943	-8.07	4.69897	4.69897
28	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1116	8.943	-8.07	4.69897	4.69897
29	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1116	8.943	-8.07	4.69897	4.69897
30	1.99	1.99	1.99	-0.379	4.4536	-0.2	-0.201	4.1116	8.943	-8.07	4.69897	4.69897

Figure 21. Porous Asphalt tab

PICP: The structural and hydrological designs of PICP are performed in this worksheet (Figure 22) using the inputs from the user interface tab and the detailed inputs tab. The design process followed for calculations was discussed in Chapter 3. This worksheet is locked and can only be unlocked with a password provided to the principal owner of the tool.



Permeable Interlocking Concrete Pavers (PICP)

CONSTRAINTS

Setbacks: Water supply wells require 100' and Septic systems require 50'
Underdrain required when bedrock is less than 2' beneath the pavement surface
Site Topography: Slope should not be greater than 2%
Soils Requirement: HSG C or D requires an underdrain
Utility Requirement: Consider clearance for all utilities. Min. 5' from down-gradient wet utility lines
Water Table Requirement: Minimum of 2' separation from base layer
All soils should have a minimum CBR of 4%

STRUCTURAL DESIGN

INPUTS

W_{18}: 50,000 R (%): 80 Z_R: -0.84162 S_0: 0.45 ΔPSI: 1.7 M_R (psi): 7157 a_1: 0.30 a_2: 0.09 a_3: 0.06 Thickness of Bedding Layer (in.): 2

OUTPUTS

SN_{Design}: 1.99 Thickness of Concrete Pavers (in.): 3.125 Thickness of Base No. 57 (in.): 4 Thickness of Subbase No. 2 (in.): 1.56 **Thickness of Subbase No. 2 (in.): 6.30

Trial	SNmin	SNavg	SNmax	A	B	C	D	E	F	G	LogW18*	LogW18
1	1.00	5.50	10.00	-0.378729555	7.60886902	-0.2	-0.2009148	0.46606889	8.94298	-8.07	7.472033	4.7
2	1.00	3.25	5.50	-0.378729555	5.88172039	-0.2	-0.2009148	0.99934761	8.94298	-8.07	5.974922	4.7
3	1.00	2.13	3.25	-0.378729555	4.6317962	-0.2	-0.2009148	3.35628098	8.94298	-8.07	4.866182	4.7
4	1.00	1.56	2.13	-0.378729555	3.82509386	-0.2	-0.2009148	8.68040951	8.94298	-8.07	4.096196	4.7
5	1.56	1.84	2.13	-0.378729555	4.24842364	-0.2	-0.2009148	5.22304022	8.94298	-8.07	4.504205	4.7
6	1.84	1.98	2.13	-0.378729555	4.44462776	-0.2	-0.2009148	4.15429341	8.94298	-8.07	4.690513	4.7
7	1.98	2.05	2.13	-0.378729555	4.53928913	-0.2	-0.2009148	3.72689959	8.94298	-8.07	4.779628	4.7
8	1.98	2.02	2.05	-0.378729555	4.49223399	-0.2	-0.2009148	3.93289857	8.94298	-8.07	4.735396	4.7
9	1.98	2.00	2.02	-0.378729555	4.46850057	-0.2	-0.2009148	4.04158999	8.94298	-8.07	4.713037	4.7
10	1.98	1.99	2.00	-0.378729555	4.45658169	-0.2	-0.2009148	4.09742958	8.94298	-8.07	4.701795	4.7
11	1.98	1.99	1.99	-0.378729555	4.45060912	-0.2	-0.2009148	4.12573211	8.94298	-8.07	4.696159	4.7
12	1.99	1.99	1.99	-0.378729555	4.4535965	-0.2	-0.2009148	4.11154867	8.94298	-8.07	4.698979	4.7
13	1.99	1.99	1.99	-0.378729555	4.45210308	-0.2	-0.2009148	4.11863233	8.94298	-8.07	4.697569	4.7
14	1.99	1.99	1.99	-0.378729555	4.45284986	-0.2	-0.2009148	4.11508849	8.94298	-8.07	4.698274	4.7
15	1.99	1.99	1.99	-0.378729555	4.4532232	-0.2	-0.2009148	4.11331808	8.94298	-8.07	4.698626	4.7
16	1.99	1.99	1.99	-0.378729555	4.45340985	-0.2	-0.2009148	4.11243325	8.94298	-8.07	4.698802	4.7
17	1.99	1.99	1.99	-0.378729555	4.45350318	-0.2	-0.2009148	4.11199093	8.94298	-8.07	4.698890	4.7
18	1.99	1.99	1.99	-0.378729555	4.45354984	-0.2	-0.2009148	4.11176979	8.94298	-8.07	4.698934	4.7
19	1.99	1.99	1.99	-0.378729555	4.45357317	-0.2	-0.2009148	4.11165923	8.94298	-8.07	4.698957	4.7
20	1.99	1.99	1.99	-0.378729555	4.45358483	-0.2	-0.2009148	4.11160395	8.94298	-8.07	4.698968	4.7
21	1.99	1.99	1.99	-0.378729555	4.45359067	-0.2	-0.2009148	4.11157631	8.94298	-8.07	4.698973	4.7
22	1.99	1.99	1.99	-0.378729555	4.45358775	-0.2	-0.2009148	4.11159013	8.94298	-8.07	4.698970	4.7
23	1.99	1.99	1.99	-0.378729555	4.45358629	-0.2	-0.2009148	4.11159704	8.94298	-8.07	4.698969	4.7
24	1.99	1.99	1.99	-0.378729555	4.45358702	-0.2	-0.2009148	4.11159359	8.94298	-8.07	4.698970	4.7
25	1.99	1.99	1.99	-0.378729555	4.45358739	-0.2	-0.2009148	4.11159186	8.94298	-8.07	4.698970	4.7
26	1.99	1.99	1.99	-0.378729555	4.45358757	-0.2	-0.2009148	4.11159099	8.94298	-8.07	4.698970	4.7
27	1.99	1.99	1.99	-0.378729555	4.45358748	-0.2	-0.2009148	4.11159143	8.94298	-8.07	4.698970	4.7
28	1.99	1.99	1.99	-0.378729555	4.45358743	-0.2	-0.2009148	4.11159164	8.94298	-8.07	4.698970	4.7
29	1.99	1.99	1.99	-0.378729555	4.45358746	-0.2	-0.2009148	4.11159153	8.94298	-8.07	4.698970	4.7
30	1.99	1.99	1.99	-0.378729555	4.45358747	-0.2	-0.2009148	4.11159148	8.94298	-8.07	4.698970	4.7

HYDROLOGICAL DESIGN

INPUTS

Pervious Area A_p (ft^2): 16000 Impervious Area A_i (ft^2): 32000 The ratio of the contributing drainage area R: 2.00 The rainfall depth for the Treatment Volume P (ft): 0.1
Infiltration rate for native soils i (ft/day): 1 The time to fill the reservoir layer t_f (day): 0.0833333 The void ratio for the reservoir layer V_r: 0.4 volumetric runoff coeff. for treatment area Rv: 0.95

OUTPUTS

Treatment Volume Tv (ft^3): 4560 The depth of runoff from the contributing drainage area dc (ft): 0.14 maximum detention time t_d (days): 1
The depth of the reservoir layer dp (in): 10.30 maximum depth of the reservoir layer dp-max (ft.): 15

User Interface | Detailed Inputs | CONCRETE | ASPHALT | PICP | Optimization

Figure 22. PICP tab

Optimization: In this worksheet shown in Figure 23, the optimization algorithm explained in Chapter 3 is implemented. The design and cost calculations are done for each optimization combination option. 10,000 design trials are performed for each option to obtain the most economical and display it in the user interface tab. This worksheet is also locked and can only be unlocked with a password provided to the principal owner of the tool.

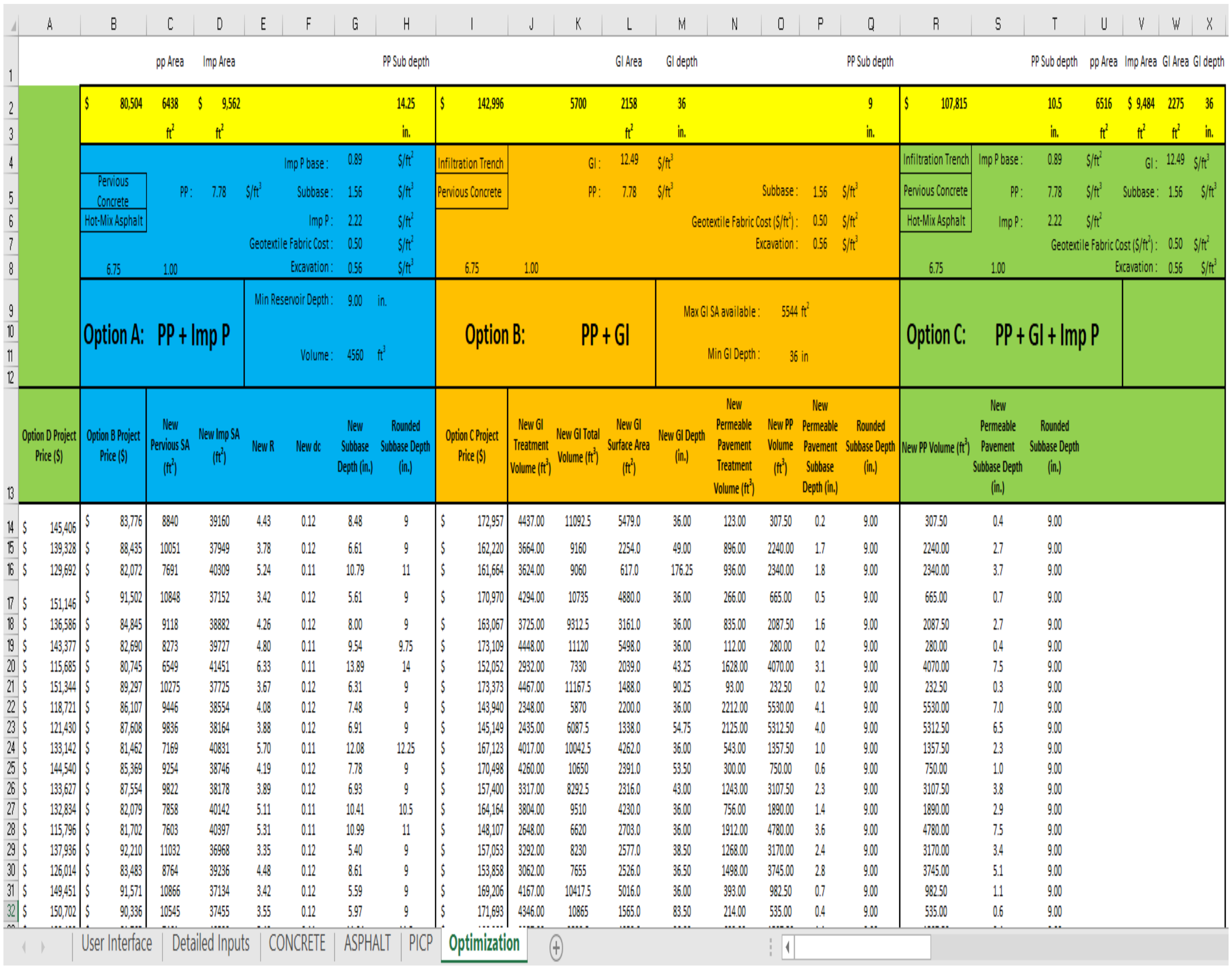


Figure 23. Optimization tab

5.2 Sensitivity Analysis Results

5.2.1 Pervious Concrete

<u>Scenario 1</u>: In this scenario, the thickness of the pervious concrete had to be determined at different traffic levels (ADTT) and soil's strengths (CBR of the soil). The results from the COTPP are shown in Table 13 and the PerviousPave results can be found in Appendix C. Figure 24 presents the summary of the comparison.

Table 13. Results from COTPP for design scenario 1 (pervious concrete)

	CBR (%)							
	4	5	6	7	8	9	10	15
AADTT	PC Thicknesses							
2	7	6.75	6.75	6.75	6.5	6.5	6.5	6.25
3	7	6.75	6.75	6.75	6.5	6.5	6.5	6.25
4	7	7	6.75	6.75	6.75	6.5	6.5	6.25
5	7	7	6.75	6.75	6.75	6.5	6.5	6.5
6	7	7	6.75	6.75	6.75	6.75	6.5	6.5
7	7	7	6.75	6.75	6.75	6.75	6.5	6.5
8	7	7	6.75	6.75	6.75	6.75	6.5	6.5
9	7	7	7	6.75	6.75	6.75	6.75	6.5
10	7	7	7	6.75	6.75	6.75	6.75	6.5

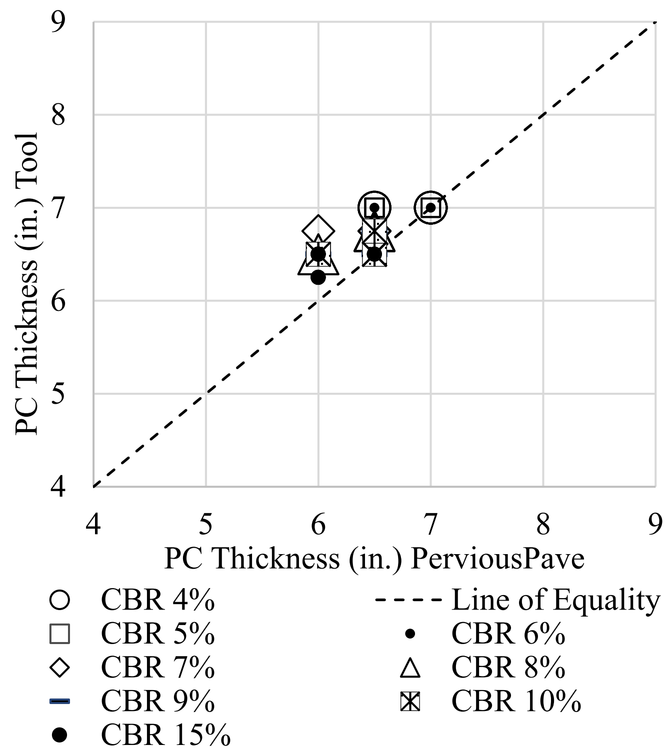


Figure 24. Comparison of pervious concrete thicknesses

The comparison was done by graphing linear inequalities where the results from PerviousPave are on the x-axis and the ones from the COTPP are on the y-axis. The dashed boundary line called the line of equality represents the linear equation x = y, which means that the outputs from both tools match. The regions on the right and left of the line represent the entire set of solutions for the inequalities x > y and y > x, respectively.

Looking at the graph in Figure 24, it can be said that all the thicknesses are close to the line of equality, which means that the values are nearly matching. The pervious concrete thicknesses from the COTPP are slightly larger than the ones from PerviousPave because most points are located on the left region of the line of equality (y > x). The formulas used in the tool to determine the composite modulus of subgrade/subbase reaction (k) and the number of load application (N) for each axle type were obtained from literature since ACPA did not provide those formulas. This is the reason why the values from the COTPP are different from PerviousPave values. The percent difference between the thickness values from both tools ranged from 0 to 11.8% with the median and the average being equal to 3.8% and 4.4%, respectively. Therefore, it is safe to state that the PC structural design results are reliable.

Scenario 2: In this scenario, the thickness of the reservoir layer or base/subbase layer had to be found based on varying soil infiltration rates and design storm precipitations. Table 14 shows the results from the COTPP and Figure 25 shows the summary of the comparison. PerviousPave results can be found in Appendix C.

Table 14. Results from COTPP for design scenario 2 (pervious concrete)

	Infiltration Rate of Subgrade (in./hr)						
	0.5	0.75	1	1.25	1.5	1.75	2
Precipitation (in.)	Reservoir Thicknesses						
1	8.5	7.75	7.25	6.5	6	6	6
1.2	10.5	9.75	9.25	8.5	8	7.2	6.75
1.25	11	10.25	9.75	9	8.5	7.75	7.25
1.5	13.25	12.75	12	11.5	10.75	10.25	9.5
2	18	17.5	16.75	16.25	15.5	15	14.25
2.25	20.5	20	19.25	18.75	18	17.5	16.75
2.5	23	22.25	21.75	21	20.5	19.75	19.25
2.75	25.25	24.75	24	23.5	22.75	22.25	21.5
3	27.75	27	26.5	25.75	25.25	24.5	24
3.25	30.25	29.5	29	28.25	27.75	27	26.5
3.5	32.5	32	31.25	30.75	30	29.5	28.75
3.75	35	34.25	33.75	33	32.5	31.75	31.25
4	37.25	36.75	36	35.5	34.75	34.25	33.5
4.25	39.75	39.25	38.5	38	37.25	36.75	36
4.5	42.25	41.5	41	40.25	39.75	39	38.5
4.75	44.5	44	43.25	42.75	42	41.5	40.75
5	47	46.75	45.75	45	44.5	43.75	43.25
5.25	49.5	48.75	48.25	47.5	47	46.25	45.75
5.5	51.75	51.25	50.5	50	49.25	48.75	48
5.75	54.25	53.5	53	52.25	51.75	51	50.5
6	56.5	56	55.25	54.75	54	53.5	52.75

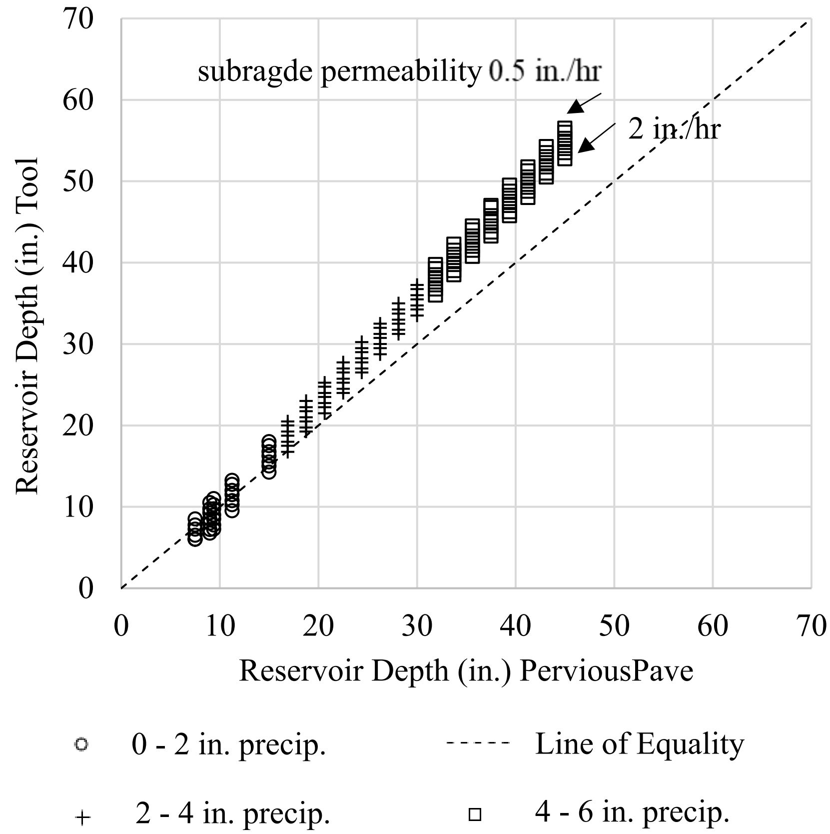


Figure 25. Comparison of base/subbase layer thicknesses

Like in design scenario 1, the reservoir thicknesses from the COTPP are slightly larger than the ones from PerviousPave because all points in Figure 25 are located on the left side of the line of equality (y > x). It can also be noticed that the reservoir thicknesses from the COTPP continue to get larger as the storm design precipitation increases. This is due to the fact that, contrarily to PerviousPave, the COTPP designs the reservoir thickness with a safety factor (infiltration rate of soil is divided by 2) and the PP surface layer is not considered for additional storage capacity. This results in the reservoir thicknesses from the COTPP being more conservative at high storm precipitations than the reservoir thicknesses from PerviousPave. The percent difference between the reservoir thickness values from both tools ranged from 0 to 22.9% with the median and the average being equal to 16.3% and 15.1%, respectively. It is important to note that PPs are usually built to store the first flush runoff, which is approximately 1-1.5 in. depending on the location in Alabama (Dylewski et al., n.d.). It can be seen in Figure 25 that the thicknesses at low rainfall depths are extremely close to the line of equality. This means that the results from the COTPP are conservative and the tool can be confidently used to design hydrologically pervious concrete pavements.

To verify that the base/subbase thicknesses from the tool are less conservative when designed without a safety factor, scenario 2 was repeated without dividing the infiltration rate by 2. The results are shown in Figure 26. The thicknesses are closer to the line of equality and even less conservative at low rainfall depths. The percent difference between the reservoir thickness values from both tools ranged from 0 to 60.9% with the median and the average being equal to 12.1% and 14.6%, respectively.

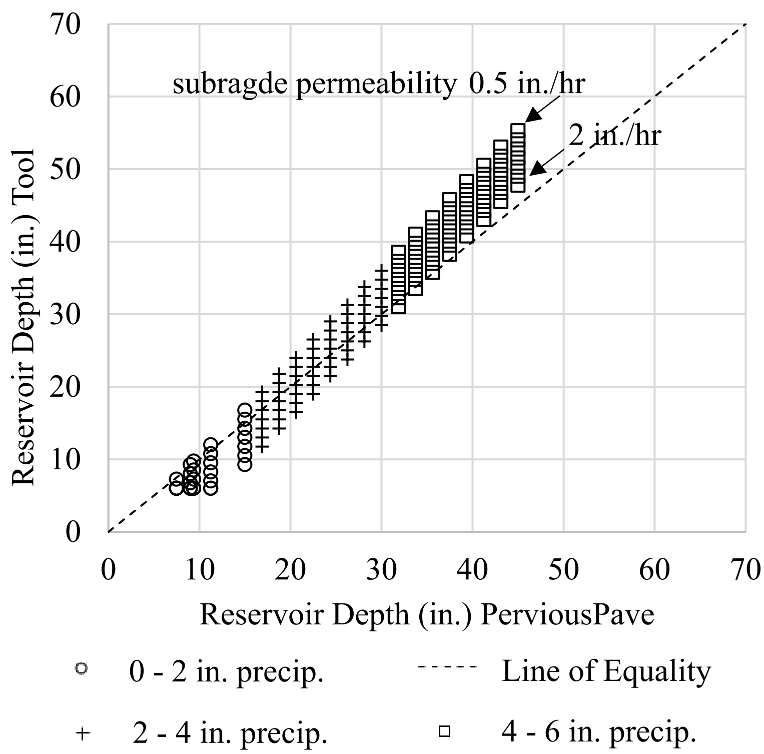


Figure 26. Comparison of base/subbase layer thicknesses (without safety factor)

5.2.2 Porous Asphalt

Appendix D shows the result tables from the NAPA guide. The results of the three scenarios from the COTPP are shown in Tables 15 through 17 and the summary of all design scenarios is shown in Figure 27.

Scenario 1: The traffic level was 27,000 ESALs and the porous asphalt thickness had to be determined based on varying base/subbase thicknesses and design subgrade resilient modulus.

Table 15. Porous asphalt thicknesses (Scenario 1)

		Design Subgrade Resilient Modulus (psi)					
		2000	3000	4000	6000	8000	10,000
Base Thickness (in)	6	7.0	6.0	5.5	4.75	4.25	4.0
	12	4.5	4.25	3.75	3.75	3.5	3.25
	18	4.0	4.0	3.5	3.5	3.25	3.25
	24	4.0	3.75	3.5	3.5	3.25	3.0
	30	3.75	3.5	3.25	3.5	3.25	3.0

Scenario 2: The traffic level was 110,000 ESALs and the porous asphalt thickness had to be obtained based on varying base/subbase thicknesses and design subgrade resilient modulus.

Table 16. Porous asphalt thicknesses (Scenario 2)

		Design Subgrade Resilient Modulus (psi)					
		2000	3000	4000	6000	8000	10,000
Base Thickness (in)	6	8.5	7.5	6.75	6	5.25	5
	12	5.5	5.25	5.0	4.75	4.50	4.25
	18	5.25	5.00	4.5	4.50	4.25	4.0
	24	5.0	4.75	4.50	4.50	4.25	4.0
	30	4.75	4.50	4.25	4.25	4.25	4.0

<u>Scenario 3</u>: The traffic level was 820,000 ESALs and the porous asphalt thickness had to be determined based on varying base/subbase thicknesses and design subgrade resilient modulus.

Table 17. Porous asphalt thicknesses (Scenario 3)

		Design Subgrade Resilient Modulus (psi)					
		2000	3000	4000	6000	8000	10,000
Base Thickness (in)	6	11.25	10	9	8	7.25	6.75
	12	7.5	7	6.75	6.5	6	5.75
	18	7	6.75	6.25	6.25	6	5.75
	24	6.75	6.5	6	6	5.75	5.5
	30	6.5	6.25	6	6	5.75	5.5

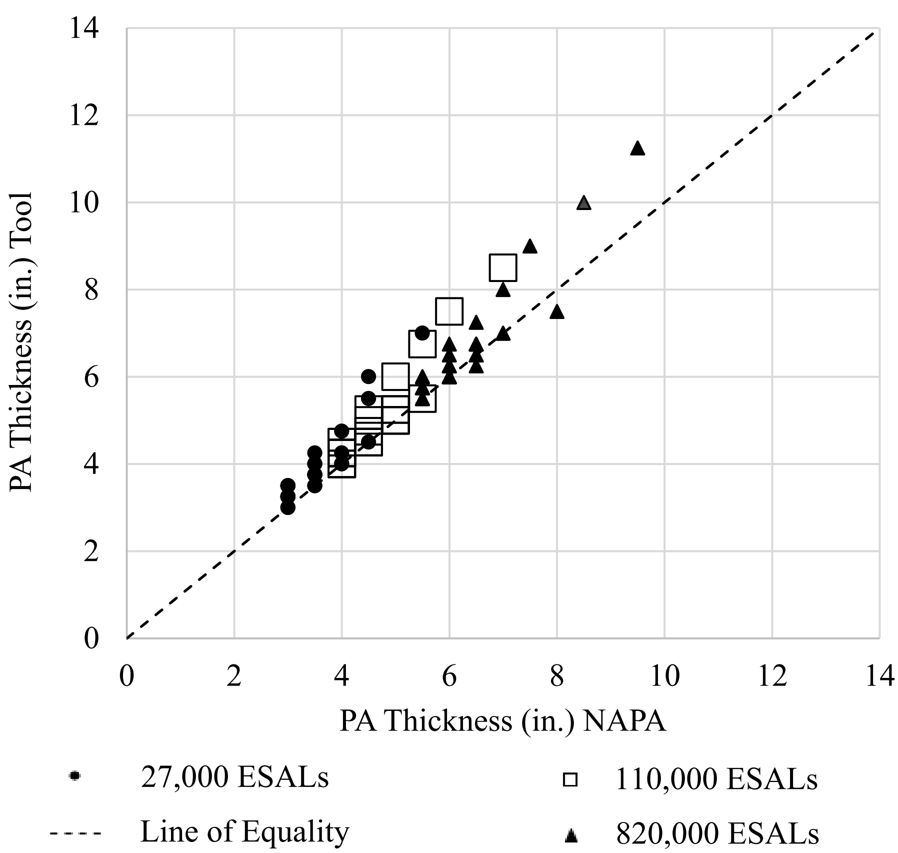


Figure 27. Comparison of porous asphalt thicknesses

It can be seen in Figure 27 that most thickness values are close to the line of equality regardless of the design ESALs level. The few values that are far from the line of equality represent the porous asphalt thicknesses when the base/subbase layer is less than 12 in. thick. Those thickness values from the COTPP are more conservative than the values provided in the NAPA guide. The reason why is that it is recommended in the NAPA guide that the porous asphalt thickness over a base/subbase layer with a thickness less than 12 in. should be designed with two methods: the composite subgrade method and the conventional 1993 method. It is recommended to use the most conservative result out of the two methods. However, it was noticed that the result tables provided in the NAPA guide only included results from the composite subgrade method (least conservative method) for 6in. thick base/subbase layer. Since porous asphalt thicknesses in the COTPP are designed with both methods, only the most conservative results (usually from the conventional 1993 method) were selected for the comparison in Figure 27. Another reason for the variations in the results is the fact that the Burmister's deflection factors F_2 were obtained manually from the graph provided by NAPA and inserted as a database in the COTPP. Since the values obtained manually are not 100% accurate, this can be considered as a source of variability. The percent differences between the results of the COTPP and the ones from the NAPA guide ranged from 0 - 28.6% for scenario 1, 0 - 22.2% for scenario 2, and 0 - 18.2% for scenario 3. The median percent differences were 8%, 5.4%, and 4.1% for scenarios 1, 2, and 3, respectively. The average percent differences were 9.8%, 6.4%, and 5.3% for design scenarios 1, 2, and 3, respectively. This gives an overall average percent difference of 7.2% and shows that the results from the COTPP are conservative. Therefore, the tool can help design PA pavements effectively.

5.2.3. Permeable Interlocking Concrete Pavers (PICP)

The surface layer, the bedding layer, and the base layer have fixed thickness values when it comes to PICP. Therefore, the design scenario consisted of finding the subbase thicknesses to help the PICP support traffic. The subbase thickness had to be found based at different traffic level (ESALs) and soil's strengths (CBR of soil). Table 18 shows the results from the COTPP and Figure 28 shows the summary of the comparison. ICPI guide results can be found in Appendix E.

Table 18. Base and Subbase thicknesses for PICP

Lifetime ESALs	CBR (%) Resilient Modulus Mr (psi)	4 6205	5 7157	6 8043	7 8877	8 9669	9 10426	10 11153
50000	Base thickness No. 57 (in)	4	4	4	4	4	4	4
	Subbase thickness No. 2 (in)	6.50	6.50	6.50	6.50	6.50	6.50	6.50
100000	Base thickness No. 57 (in)	4	4	4	4	4	4	4
	Subbase thickness No. 2 (in)	8	6.5	6.5	6.5	6.5	6.5	6.5
200000	Base thickness No. 57 (in)	4	4	4	4	4	4	4
	Subbase thickness No. 2 (in)	12.75	10.25	8.5	7	6.5	6.5	6.5
300000	Base thickness No. 57 (in)	4	4	4	4	4	4	4
	Subbase thickness No. 2 (in)	15.75	13.25	11.25	9.75	8.5	7.25	6.5
400000	Base thickness No. 57 (in)	4	4	4	4	4	4	4
	Subbase thickness No. 2 (in)	18.25	15.5	13.5	11.75	10.25	9.25	8
500000	Base thickness No. 57 (in)	4	4	4	4	4	4	4
	Subbase thickness No. 2 (in)	20	17.25	15	13.25	12	10.75	9.5
600000	Base thickness No. 57 (in)	4	4	4	4	4	4	4
	Subbase thickness No. 2 (in)	21.5	18.75	16.5	14.75	13.25	12	10.75
700000	Base thickness No. 57 (in)	4	4	4	4	4	4	4
	Subbase thickness No. 2 (in)	23	20	17.75	16	14.5	13	12
800000	Base thickness No. 57 (in)	4	4	4	4	4	4	4
	Subbase thickness No. 2 (in)	24.25	21.25	19	17	15.5	14	13
900000	Base thickness No. 57 (in)	4	4	4	4	4	4	4
	Subbase thickness No. 2 (in)	25.25	22.25	20	18	16.25	15	13.75
1000000	Base thickness No. 57 (in)	4	4	4	4	4	4	4
	Subbase thickness No. 2 (in)	26.25	23.25	20.75	18.75	17.25	15.75	14.5

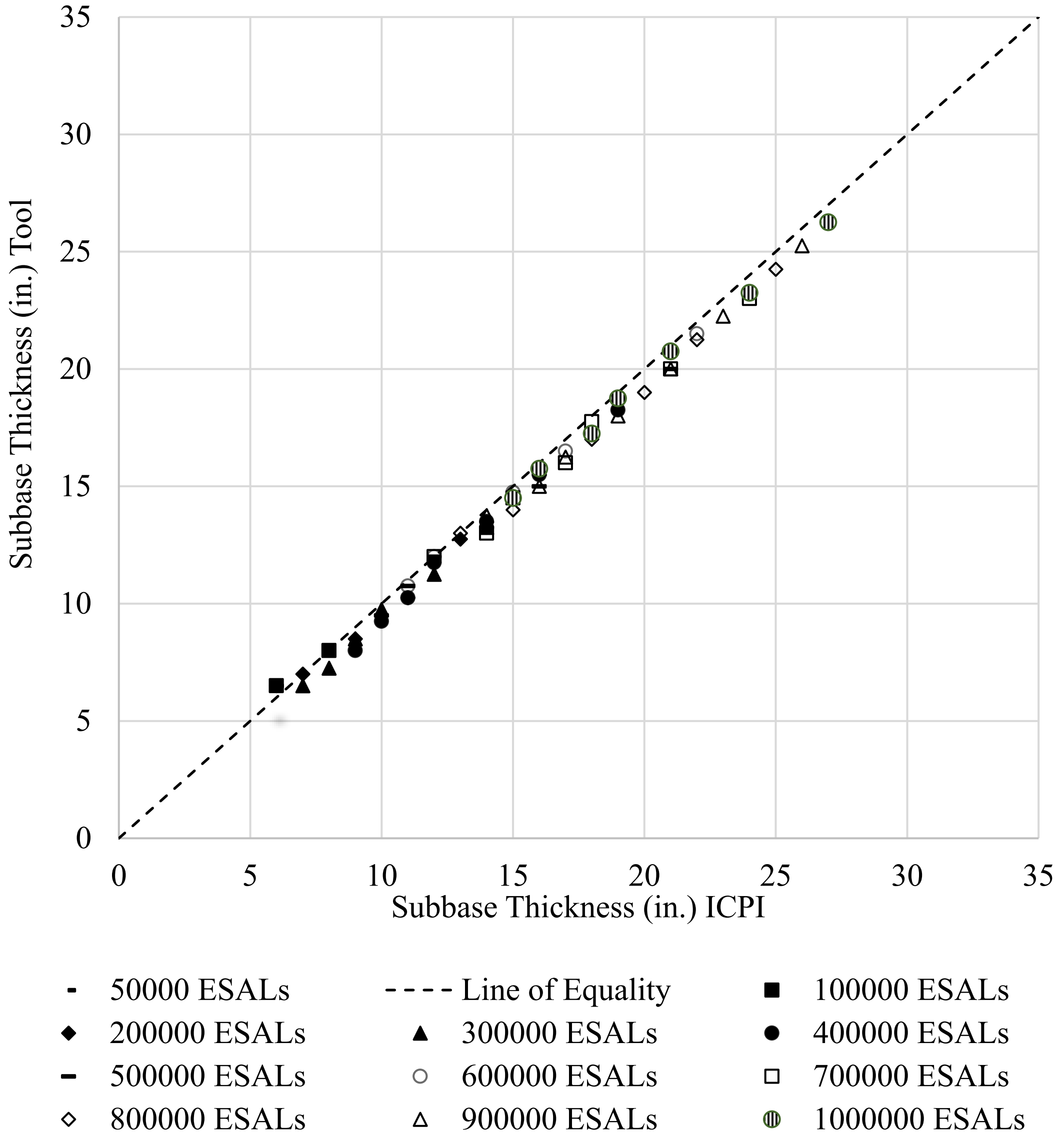


Figure 28. Comparison for base and subbase thicknesses (PICP)

Since the base thickness No. 57 is always equal to 4 inches, only the subbase thicknesses No. 2 are used for the comparison, which is summarized in Figure 28. It can be seen in Figure 28 that almost all points are extremely close to the line of equality, which means that the recommendations from the ICPI guidelines are not remarkably different from the results of the COTPP. The percent differences between the results ranged from 0 to 11.8% with the median and the average being equal to 4.9% and 4.8%, respectively. A possible reason why certain points are not exactly on the line of equality might be the absence of some assumed inputs such as the standard deviation (S_0) and the ΔPSI from the ICPI guidelines. As a result, the AASHTO recommended value of 0.45 for standard deviation and an assumed value of 1.7 for ΔPSI were used for the design in the COTPP. The results of PICP design from the COTPP are the least conservative compared to other types of PP. However, it is important to note that the bedding layer, which is not considered in the design process, helps increase structural capacity and can help make the results more conservative. From this comparison, it can be said that the PICP structural design from the COTPP is acceptable.

Overall, the structural and hydrological design results from the COTPP are satisfying and demonstrate that the results from tool can be trusted.

5.3 Case Study Results

Using the general inputs, the exact match of the PICP cross section at the existing site was obtained from the COTPP as shown in Figure 29. It is important to note that the existing PICP area (7800 ft^2) occupies 21% of the parking lot, HMA (29,250 ft^2) occupies 79% of parking lot, and the bioretention treats approximately 79% of total treatment volume. All these values were inputted in the COTPP to ensure that the resulting total cost for the combination of PICP, HMA, and bioretention from the tool was equal the actual final cost of $305,918. The details concerning the inputs can be found in Appendix F and the results are shown in Figure 30. The final cost obtained from the COTPP was $304,320, which means there is a difference of $1,598. This difference is due to the fact that certain values are slightly not matched. For instance, it can be seen in Figure 30 that the PICP area was 7,781 ft^2 instead of 7,800 ft^2. As a result, every cost generated by the COTPP was corrected for comparison by adding the $1,598 difference.

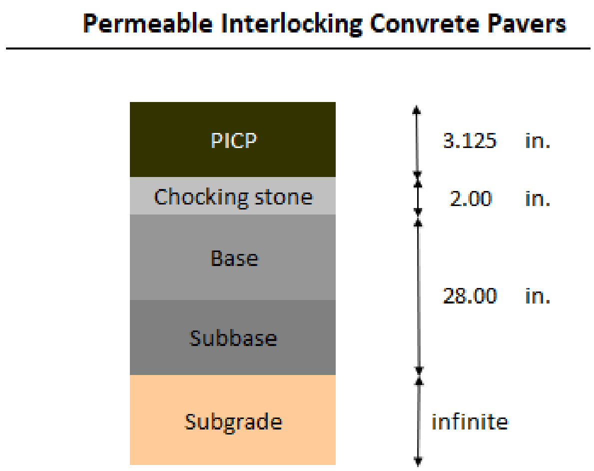


Figure 29. Case study site - PICP cross section from COTPP

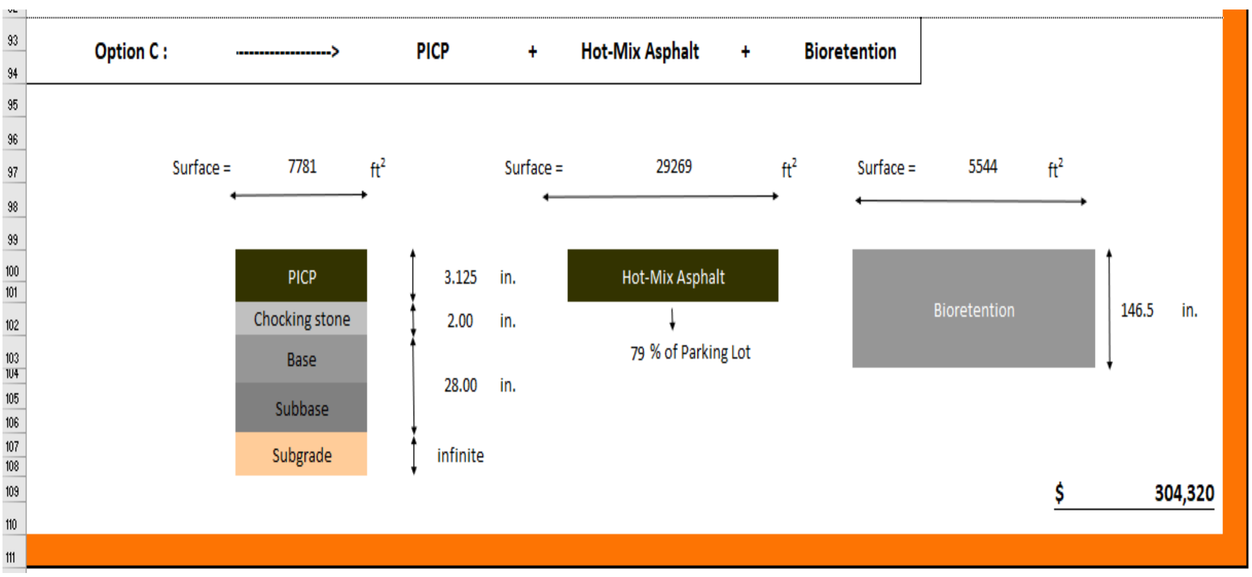


Figure 30. Reference example to match actual cost of case study site

<u>Scenario 1</u>: To compare to the actual construction cost ($305,918) of the studied site, final costs had to be determined from the COTPP optimization process at different PICP areas. Since PICP occupies 21% of existing parking lot, half of that value (10.5% or around 11%) was used as the minimum PICP area and 26% was used as the maximum area. The area was increased at 1%

increment. The results from the COTPP are shown in Table 19 and the detailed inputs can be found in Appendix F. Figure 31 displays the summary of the comparison.

Table 19. Construction costs from COTPP (Scenario 1)

PICP (% Area of Parking Lot)	Bioretention (% of Treatment Volume)	Cost ($)	Corrected Cost ($)
11	96.5	237,538	239,136
12	96.8	239,875	241,473
13	96.5	245,008	246,606
14	95.5	250,111	251,709
15	95.3	257,861	259,459
16	94.8	260,966	262,564
17	94.7	267,404	269,002
18	94.1	277,252	278,850
19	93.8	281,532	283,130
20	93.7	282,545	284,143
21	93.1	287,890	289,488
22	92.0	293,558	295,156
23	92.5	299,911	301,509
24	92.3	304,072	305,670
25	92.2	309,326	310,924
26	91.2	314,773	316,371

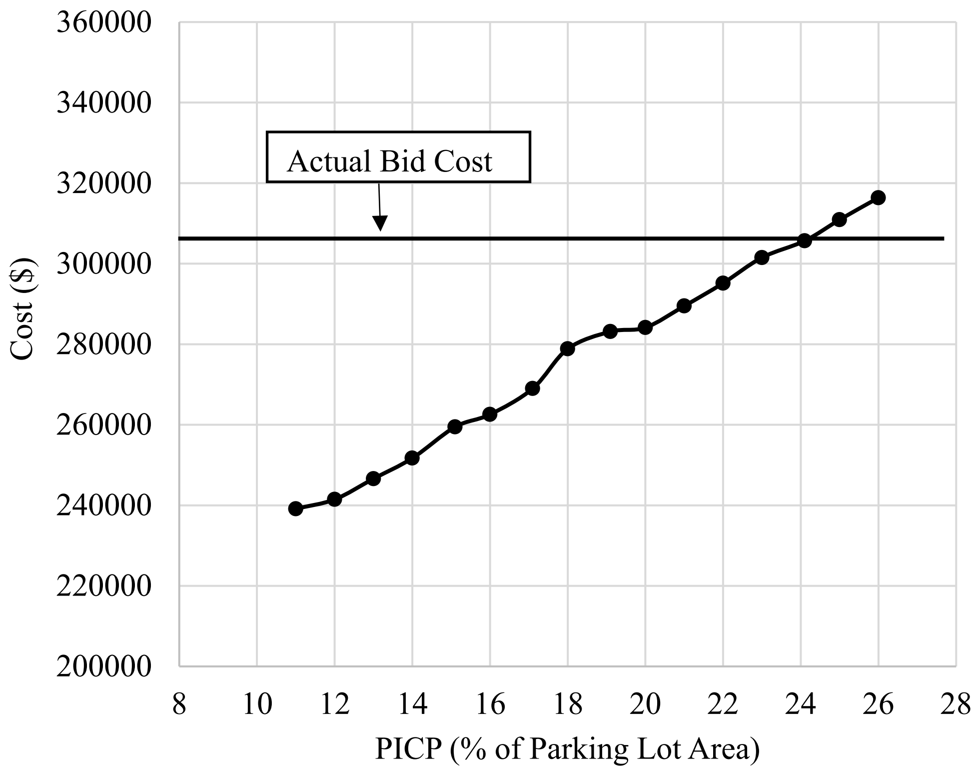


Figure 31. Construction costs comparison (Scenario 1)

Figure 31 shows that cost increases as the PICP area is increased. This is due to the high cost of PICP (15.38 $/ft^2) obtained for the case study. This cost is unusual, but it might include prices for other components like an underdrain. The construction costs were lower than $305,918 at PICP areas ranging from 11% to 24% of parking lot area and higher than $305,918 at PICP areas above 24% of the parking lot. The COTPP was even able to optimize cost even when the PICP area was 21% of parking lot area, which is the original PICP area at the existing site. This was achieved by the COTPP via adjustment of the original parking lot design. Although these values are satisfying, it can be seen in Table 19 that the bioretention must treat more than 90% of total treatment volume to achieve optimization. To ensure that it does not always have to be the case, a second scenario was simulated in the COTPP.

Scenario 2: In this scenario, construction costs had to be determined at different amount levels of treatment volume to be treated by the bioretention and compared to $305,918 to verify optimization. First, the area of PICP was maintained at 11% of the parking lot area, which is the minimum area used in Scenario 1. Second, the PICP area was kept at 21% of the parking lot area, which matches the existing PICP area at the site. Table 20 shows the results from the COTPP and the detailed inputs can be found in Appendix F. Figure 32 displays the summary of the comparison.

Table 20. Construction costs from COTPP (scenario 2)

	PICP is 11% Area of Parking Lot		PICP is 21% Area of Parking Lot	
Bioretention (% of Treatment Volume)	Cost ($)	Corrected Cost ($)	Cost ($)	Corrected Cost ($)
10	287,800	289,398	384,406	386,004
15	283,126	284,724	378,395	379,993
20	279,617	281,215	372,762	374,360
25	277,312	278,910	366,748	368,346
30	274,892	276,490	361,115	362,713
35	272,859	274,457	355,257	356,855
40	268,333	269,931	349,471	351,069
45	266,234	267,832	343,547	345,145
50	263,155	264,753	337,827	339,425
55	261,016	262,614	332,194	333,792
60	260,112	261,710	326,183	327,781
65	254,969	256,567	320,548	322,146
70	252,878	254,476	314,536	316,134
75	249,069	250,667	308,903	310,501
80	243,749	245,347	302,892	304,490
85	240,693	242,291	297,259	298,857
90	237,192	238,790	291,247	292,845

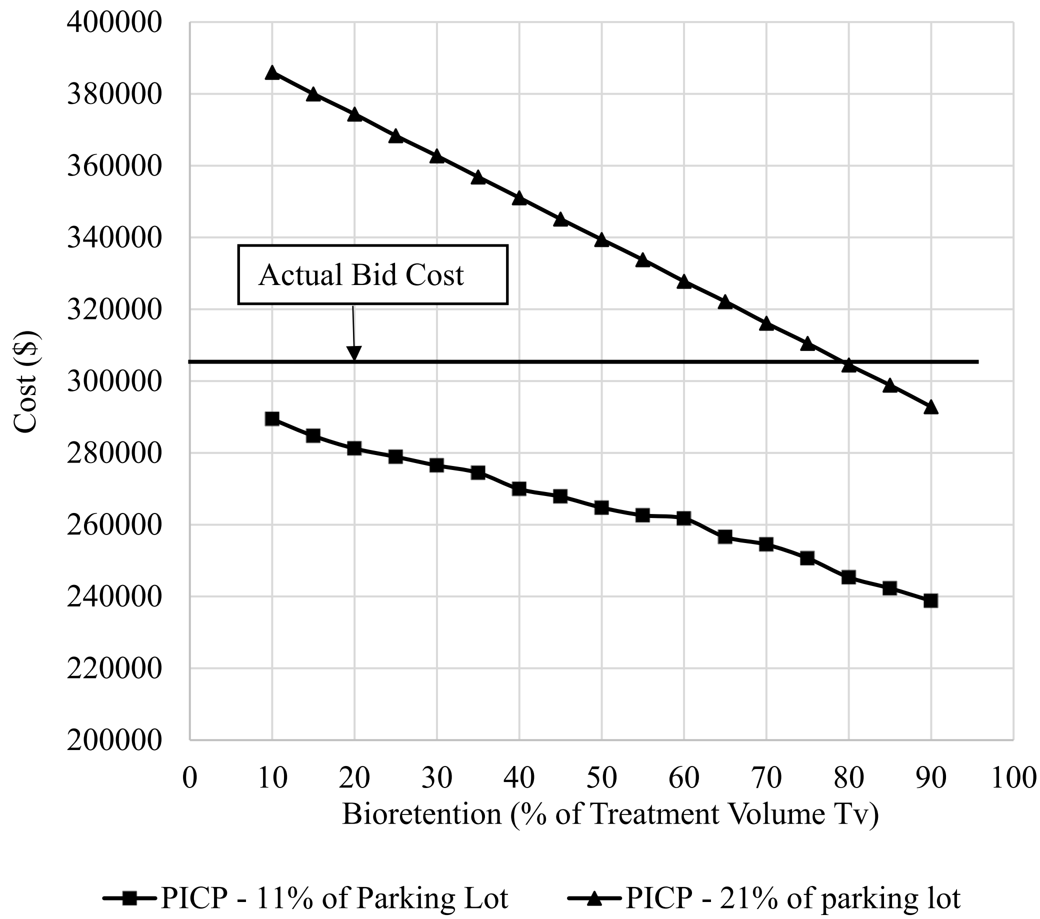


Figure 32. Construction costs comparison (Scenario 2)

From Figure 32, it can be seen that all construction costs obtained when the PICP occupies 11% of the parking lot area were lower than $305,918. However, most of the construction costs when the PCIP occupies 21% of the parking lot area were greater than $305,918. The COTPP successfully optimized the costs when the bioretention treated 80% or more of the total treatment volume. This means that, for the specific selected site, construction costs are lowered when the PICP area is reduced and the bioretention treatment volume is increased. The lowest construction cost was $238,790 at PICP area occupying 11% of parking lot area and bioretention treating 90% of total treatment volume.

Based on the results from the case study, it can be affirmed that the final cost of the existing parking lot can be reduced from $305,918 to $238,790. This demonstrates that the COTPP has the capacity to optimize costs of permeable pavements. Moreover, the dimensioning for optimization of individual components (aggregate base, PC, PA etc.) that compose the parking lot significantly depends on their unit costs and design constraints. This means that the most expensive component will control the design and optimization process.

CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

6.1 Research Conclusions

The Cost Optimization Tool for Permeable Pavements (COTPP) was developed to design every type of permeable pavement and optimize their construction costs. The Tool contains three unique design methods for the three types of permeable pavements, which are PC, PA, and PICP. All permeable pavements have the same hydrological design method (ICPI method), but different structural design methods (mechanistic design methodology and AASHTO 1993). All these methods and costs of each type of permeable pavement are combined for the first time in a tool to help designers in their design process and decision making. The costs are optimized through the combination of GIs to choose the most cost-effective design option and compete with conventional pavement systems. The COTPP was developed in Microsoft Excel for easy access and utilization by design engineers.

In a sensitivity analysis of the tool, the results from the structural and hydrological design of PPs were compared to results from the software PerviousPave and design guidelines from associations such as NAPA and ICPI. The average percent differences between the structural design results were 4.4%, 7.2%, and 4.8% for PC, PA, and PICP, respectively. For the hydrological design results, the percent difference was 15.1% and was considered high because the COTPP uses a factor of safety that leads to more conservative results. It was concluded that the design results from the COTPP are acceptable and can be trusted in the design process of PPs.

In a case study, the construction costs of an existing parking lot located in Alabama were used in the COTPP to determine whether the tool can truly optimize costs of permeable pavements. The existing parking lot site included PICP, a bioretention, and a HMA pavement. It was found that the actual cost of the parking lot ($305,918) could be reduced to as low as $238,790. Through those results, it was verified that the COTPP developed for Alabama designers and engineers during their planning stage could effectively optimize cost of permeable pavements. The controlling factors in the optimization process are the unit costs of individual components (aggregate base, PC, PA etc.) that compose the parking lot and design constraints.

6.2 Recommendations for Future Research

As recommendations for the advancement of research concerning the design of permeable pavements and the optimization of their costs, the following ideas can be considered:

1. Add Level 1 design procedure to the tool for the addition of underdrains when infiltration rates of soils are below the recommended minimum (0.5 in./hr.) or to allow water treatment volume in excess to overflow into existing stormwater conveyance networks.
2. Determine the actual curve number of PPs for a more accurate hydrological design.
3. Determine the structural material properties of PPs.
4. Develop a unique structural design method for all types of PP that does not rely on any conventional pavements' properties but takes into consideration the different modes of failure of PPs and a validated model of their performance.
5. Determine current costs of all PPs' components.
6. Include life-cycle cost analysis (LCCA) in the COTPP to account for all costs from land cost to maintenance and rehabilitation costs.
7. Combine the COTPP from this study to the practical tool developed by Ellis (2020) to obtain a single tool. That tool will execute the design of the three GIs (PP, bioretention, and infiltration trench) in various ways: (1) individually to treat runoff from all types of land including impervious conventional pavements; (2) in combination between themselves; (3) in combination with a BMP which is a detention pond.

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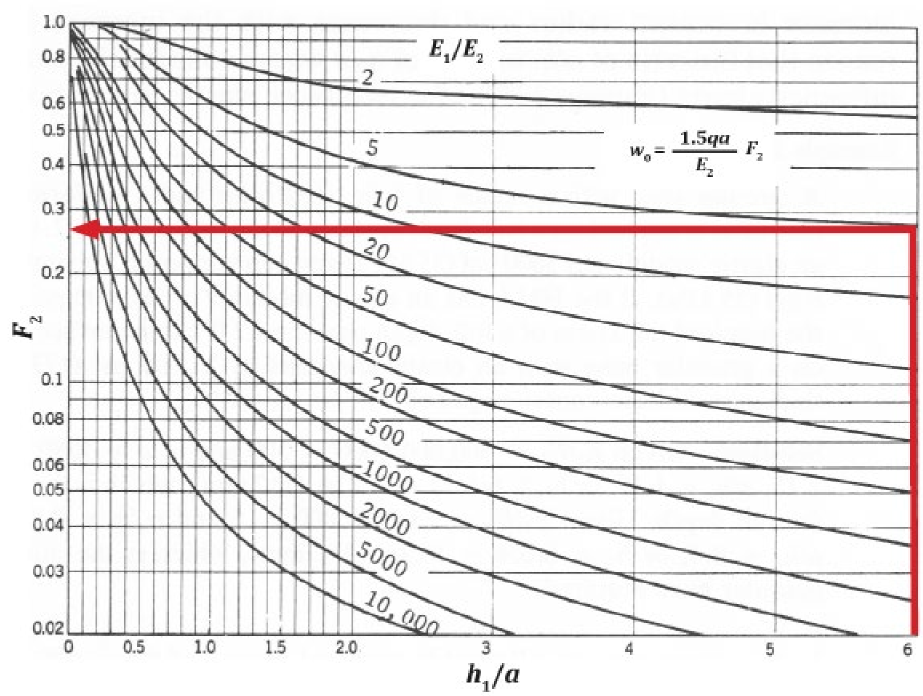
APPENDIX A: Database for Burmister's Deflection Factor (F_2)

The Burmister's Deflection Factor F_2 is manually found from the graph shown below. To find and use this value automatically in the COTPP, the values of F_2 were obtained from the graph by hand to create a database shown in the following tables and inserted in the COTPP.



	E_1/E_2											
h1/a		0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1
	2	0	0	1	0.99	0.97	0.92	0.9	0.89	0.85	0.83	0.8
	5	0	0	1	0.92	0.88	0.81	0.77	0.71	0.68	0.63	0.6
	10	0	0	0	0	0.81	0.72	0.67	0.6	0.56	0.51	0.49
	20	0	0	0	0	0.72	0.65	0.58	0.5	0.47	0.42	0.4
	50	1	0.9	0.8	0.7	0.6	0.5	0.43	0.39	0.35	0.315	0.3
	100	0.95	0.85	0.75	0.63	0.52	0.41	0.37	0.31	0.28	0.26	0.24
	200	0.93	0.82	0.68	0.51	0.4	0.35	0.29	0.27	0.23	0.2	0.19
	500	0.91	0.75	0.6	0.42	0.31	0.27	0.22	0.19	0.175	0.16	0.15
	1000	0	0.7	0.5	0.35	0.27	0.21	0.18	0.16	0.145	0.13	0.115
	2000	0	0.55	0.4	0.27	0.21	0.18	0.15	0.14	0.115	0.098	0.085
	5000	0	0.42	0.28	0.2	0.16	0.13	0.1	0.09	0.08	0.07	0.062
	10000	0.71	0.35	0.2	0.16	0.12	0.092	0.08	0.065	0.059	0.05	0.045

h1/a \ E_1 / E_2	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2
2	0.79	0.77	0.75	0.72	0.71	0.7	0.695	0.68	0.68	0.67
5	0.58	0.55	0.52	0.5	0.49	0.48	0.46	0.44	0.42	0.41
10	0.45	0.42	0.4	0.39	0.38	0.36	0.35	0.33	0.31	0.3
20	0.375	0.35	0.32	0.3	0.29	0.28	0.27	0.26	0.25	0.24
50	0.28	0.26	0.245	0.23	0.21	0.2	0.19	0.19	0.18	0.17
100	0.215	0.2	0.185	0.175	0.17	0.16	0.15	0.145	0.14	0.135
200	0.18	0.17	0.15	0.15	0.14	0.13	0.12	0.11	0.1	0.097
500	0.135	0.12	0.11	0.1	0.092	0.088	0.082	0.089	0.075	0.071
1000	0.1	0.092	0.085	0.08	0.075	0.07	0.068	0.063	0.06	0.0575
2000	0.08	0.072	0.068	0.062	0.059	0.054	0.05	0.048	0.045	0.043
5000	0.057	0.051	0.0485	0.045	0.041	0.039	0.038	0.036	0.0335	0.0315
10000	0.04	0.038	0.036	0.033	0.031	0.0295	0.0285	0.027	0.026	0.025

h1/a \ E_1 / E_2	2.1	2.2	2.3	2.4	2.5	2.6	2.7	2.8	2.9	3
2	0.666	0.662	0.658	0.654	0.65	0.646	0.642	0.638	0.634	0.63
5	0.404	0.398	0.392	0.386	0.38	0.374	0.368	0.362	0.356	0.35
10	0.295	0.29	0.285	0.28	0.275	0.27	0.265	0.26	0.255	0.25
20	0.234	0.228	0.222	0.216	0.21	0.204	0.198	0.192	0.186	0.18
50	0.166	0.162	0.158	0.154	0.15	0.146	0.142	0.138	0.134	0.13
100	0.1303	0.1256	0.1209	0.1162	0.1115	0.1068	0.1021	0.0974	0.0927	0.088
200	0.09415	0.0913	0.08845	0.0856	0.08275	0.0799	0.07705	0.0742	0.07135	0.0685
500	0.0688	0.0666	0.0644	0.0622	0.06	0.0578	0.0556	0.0534	0.0512	0.049
1000	0.05565	0.0538	0.05195	0.0501	0.04825	0.0464	0.04455	0.0427	0.04085	0.039
2000	0.04165	0.0403	0.03895	0.0376	0.03625	0.0349	0.03355	0.0322	0.03085	0.0295
5000	0.03045	0.0294	0.02835	0.0273	0.02625	0.0252	0.02415	0.0231	0.02205	0.021
10000	0.0225	0.02	0.0175	0.015	0.0125	0.01	0.0075	0.005	0.0025	0

h1/a \ E_1 / E_2	3.1	3.2	3.3	3.4	3.5	3.6	3.7	3.8	3.9	4
2	0.627	0.624	0.621	0.618	0.615	0.612	0.609	0.606	0.603	0.6
5	0.346	0.342	0.338	0.334	0.33	0.326	0.322	0.318	0.314	0.31
10	0.246	0.242	0.238	0.234	0.23	0.226	0.222	0.218	0.214	0.21
20	0.177	0.174	0.171	0.168	0.165	0.162	0.159	0.156	0.153	0.15
50	0.1266	0.1232	0.1198	0.1164	0.113	0.1096	0.1062	0.1028	0.0994	0.096
100	0.0862	0.0844	0.0826	0.0808	0.079	0.0772	0.0754	0.0736	0.0718	0.07
200	0.06685	0.0652	0.06355	0.0619	0.06025	0.0586	0.05695	0.0553	0.05365	0.052
500	0.0479	0.0468	0.0457	0.0446	0.0435	0.0424	0.0413	0.0402	0.0391	0.038
1000	0.038	0.037	0.036	0.035	0.034	0.033	0.032	0.031	0.03	0.029
2000	0.02885	0.0282	0.02755	0.0269	0.02625	0.0256	0.02495	0.0243	0.02365	0.023
5000	0.0189	0.0168	0.0147	0.0126	0.0105	0.0084	0.0063	0.0042	0.0021	0
10000	0	0	0	0	0	0	0	0	0	0

h1/a	E_1 / E_2	4.1	4.2	4.3	4.4	4.5	4.6	4.7	4.8	4.9	5
	2	0.599	0.598	0.597	0.596	0.595	0.594	0.593	0.592	0.591	0.59
	5	0.308	0.306	0.304	0.302	0.3	0.298	0.296	0.294	0.292	0.29
	10	0.208	0.206	0.204	0.202	0.2	0.198	0.196	0.194	0.192	0.19
	20	0.148	0.146	0.144	0.142	0.14	0.138	0.136	0.134	0.132	0.13
	50	0.0945	0.093	0.0915	0.09	0.0885	0.087	0.0855	0.084	0.0825	0.081
	100	0.0689	0.0678	0.0667	0.0656	0.0645	0.0634	0.0623	0.0612	0.0601	0.059
	200	0.05105	0.0501	0.04915	0.0482	0.04725	0.0463	0.04535	0.0444	0.04345	0.0425
	500	0.0372	0.0364	0.0356	0.0348	0.034	0.0332	0.0324	0.0316	0.0308	0.03
	1000	0.0284	0.0278	0.0272	0.0266	0.026	0.0254	0.0248	0.0242	0.0236	0.023
	2000	0.0207	0.0184	0.0161	0.0138	0.0115	0.0092	0.0069	0.0046	0.0023	0
	5000	0	0	0	0	0	0	0	0	0	0
	10000	0	0	0	0	0	0	0	0	0	0

h1/a	E_1 / E_2	5.1	5.2	5.3	5.4	5.5	5.6	5.7	5.8	5.9	6
	2	0.587	0.584	0.581	0.578	0.575	0.572	0.569	0.566	0.563	0.56
	5	0.289	0.288	0.287	0.286	0.285	0.284	0.283	0.282	0.281	0.28
	10	0.1895	0.189	0.1885	0.188	0.1875	0.187	0.1865	0.186	0.1855	0.185
	20	0.1285	0.127	0.1255	0.124	0.1225	0.121	0.1195	0.118	0.1165	0.115
	50	0.0799	0.0788	0.0777	0.0766	0.0755	0.0744	0.0733	0.0722	0.0711	0.07
	100	0.0581	0.0572	0.0563	0.0554	0.0545	0.0536	0.0527	0.0518	0.0509	0.05
	200	0.04195	0.0414	0.04085	0.0403	0.03975	0.0392	0.03865	0.0381	0.03755	0.037
	500	0.0296	0.0292	0.0288	0.0284	0.028	0.0276	0.0272	0.0268	0.0264	0.026
	1000	0.0207	0.0184	0.0161	0.0138	0.0115	0.0092	0.0069	0.0046	0.0023	0
	2000	0	0	0	0	0	0	0	0	0	0
	5000	0	0	0	0	0	0	0	0	0	0
	10000	0	0	0	0	0	0	0	0	0	0

APPENDIX B: Construction Costs Calculations for COTPP

The costs of excavation, geotextile fabric and PICP were provided in 2019 dollars by Wood & Volkert (2019). Before inserting them in the COTPP, the costs shown in the Figure below were converted as follows:

$$Excavation = \left(\frac{\$\,15}{CYS}\right)\left(\frac{1\ CYS}{27\ ft^3}\right) = 0.56\ \$/ft^3$$

$$Geotextile = \left(\frac{\$\,4.50}{SYS}\right)\left(\frac{1\ SYS}{9\ ft^2}\right) = 0.50\ \$/ft^2$$

$$PICP = \left(\frac{\$\,45}{SYS}\right)\left(\frac{1\ SYS}{9\ ft^2}\right) = 5\ \$/ft^2$$ Note: PICP is 3.125” thick.

Preliminary Estimate of Probable GI Construction Cost - (01/2019)

DESCRIPTION	UNIT	ESTIMATED QUANTITY	UNIT PRICE	TOTAL PRICE FOR ITEM
Project Sign	EA	1	$1,000.00	$1,000.00
Excavation, Subgrade, for Stormwater Storage Cells	CYS	1,904	$15.00	$28,555.56
Coarse Aggregate 4" Stone Paver Base, Washed	TON	236	$17.00	$4,012.00
Coarse Aggregate 10" Washed for Standard Section	TON	639	$35.00	$22,370.83
Coarse Aggregate 2' Washed for Stormwater Storage	TON	2,301	$50.00	$115,050.00
Geotextile Separation Fabric	SYS	1,311	$4.50	$5,900.00
Porous Brick Pavers - parking	SYS	1,311	$45.00	$59,000.00

The costs of excavation, drainage system, base course for conventional pavement, hot-mix-asphalt (HMA), and Portland cement concrete (PCC) were provided in 2018 dollars by Rehan et al. (2018). Before inserting them in the COTPP, the costs shown in the Figure below were converted as follows:

$$Excavation = \left(\frac{\$\,14}{CYS}\right)\left(\frac{1\ CYS}{27\ ft^3}\right) = 0.52\ \$/ft^3$$ The excavation cost 0.56 $\$/ft^3$ obtained above was used in the COTPP because it is the most conservative value.

$$Base\ course = \left(\frac{\$\,8}{SYS}\right)\left(\frac{1\ SYS}{9\ ft^2}\right) = 0.89\ \$/ft^2$$ Note: Base course is 6” thick.

$$HMA = \left(\frac{\$\,20}{SYS}\right)\left(\frac{1\ SYS}{9\ ft^2}\right) = 2.22\ \$/ft^2$$ Note: HMA is 3.75” thick.

$$PCC = \left(\frac{\$\,30}{SYS}\right)\left(\frac{1\ SYS}{9\ ft^2}\right) = 3.33\ \$/ft^2$$ Note: PCC is 6” thick.

Item Description	Unit	Unit Price	Quantity	Total
Excavation & Earth Work	CU YD	$14	1344	$18,900
Drainage System	EACH	$7,500	1	$7,500
Base Course at 6"	SQ YD	$8	4840	$38,800
HMA at 3.75"	SQ YD	$20	4840	$96,800
PCC at 6"	SQ YD	$30	4840	$145,200

The costs of interlocking concrete pavers (ICP) were provided in 2001 dollars from Walsh & Smallridge (2001). Before inserting them in the COTPP, the costs shown in the Figure below were converted as follows:

$$ICP = \left(\frac{\$\ 32.50}{m^2}\right)\left(\frac{1\ m^2}{10.76\ ft^2}\right) = 3.02\ \$/ft^2$$

PROJECT REGION	4" ICP COST $ PER M^2
GULF COAST	32.50

The costs of base course for permeable pavement, porous asphalt (PA), and pervious concrete (PC) were provided in 2018 dollars by Rehan et al. (2018). Before inserting them in the COTPP, the costs shown in the Figure below were converted as follows:

$$Base\ course = \frac{\left(\frac{\$\ 14}{SYS}\right)\left(\frac{1\ SYS}{9\ ft^2}\right)}{(12")(\frac{1\ ft}{12"})} = 1.56\ \$/ft^3$$

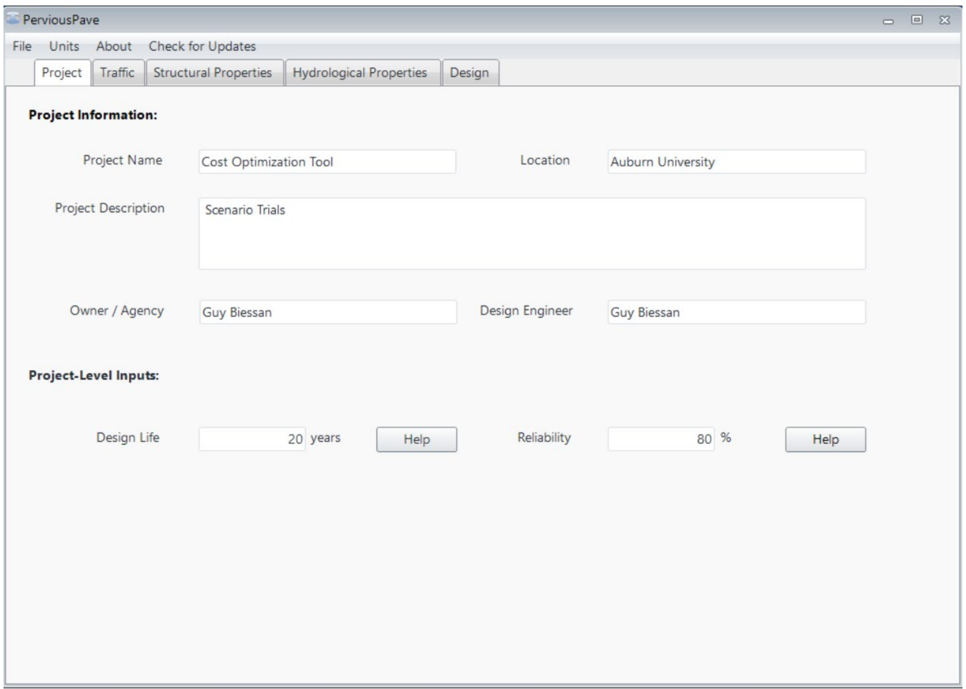
$$PA = \frac{\left(\frac{\$\ 20}{SYS}\right)\left(\frac{1\ SYS}{9\ ft^2}\right)}{(5")(\frac{1\ ft}{12"})} = 5.33\ \$/ft^3$$

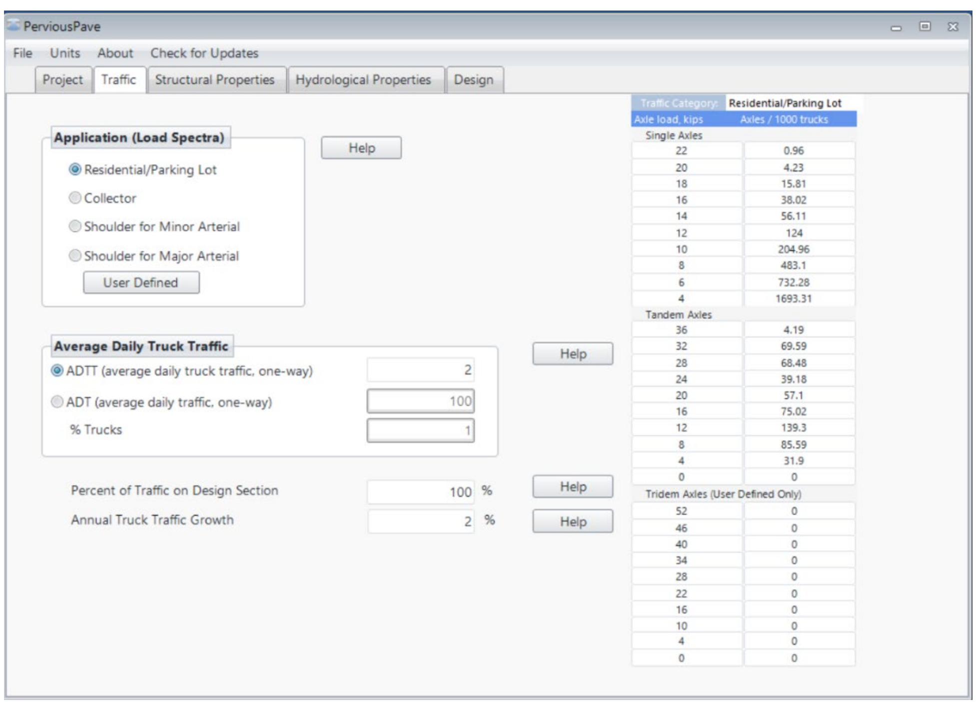
$$PC = \frac{\left(\frac{\$\ 35}{SYS}\right)\left(\frac{1\ SYS}{9\ ft^2}\right)}{(6")(\frac{1\ ft}{12"})} = 7.78\ \$/ft^3$$

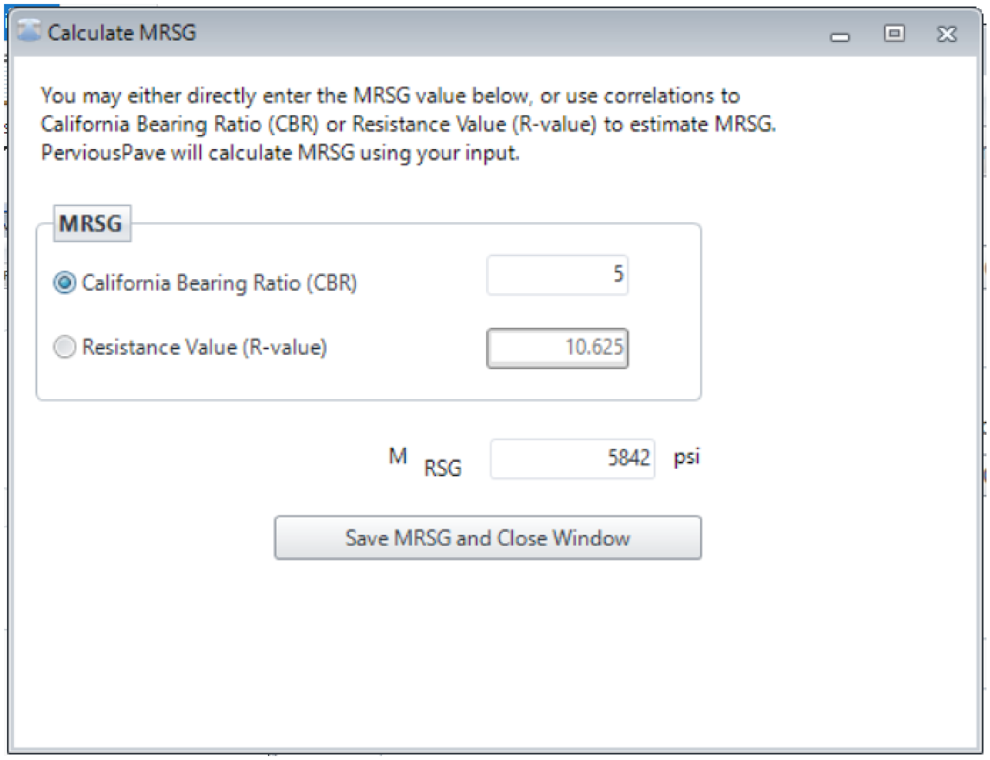
Item Description	Unit	Unit Price	Quantity	Total
Base Course at 12"	SQ YD	$14	4840	$67,800
Porous Asphalt at 5"	SQ YD	$20	4840	$96,800
Pervious Concrete at 6"	SQ YD	$35	4,840	$169,400

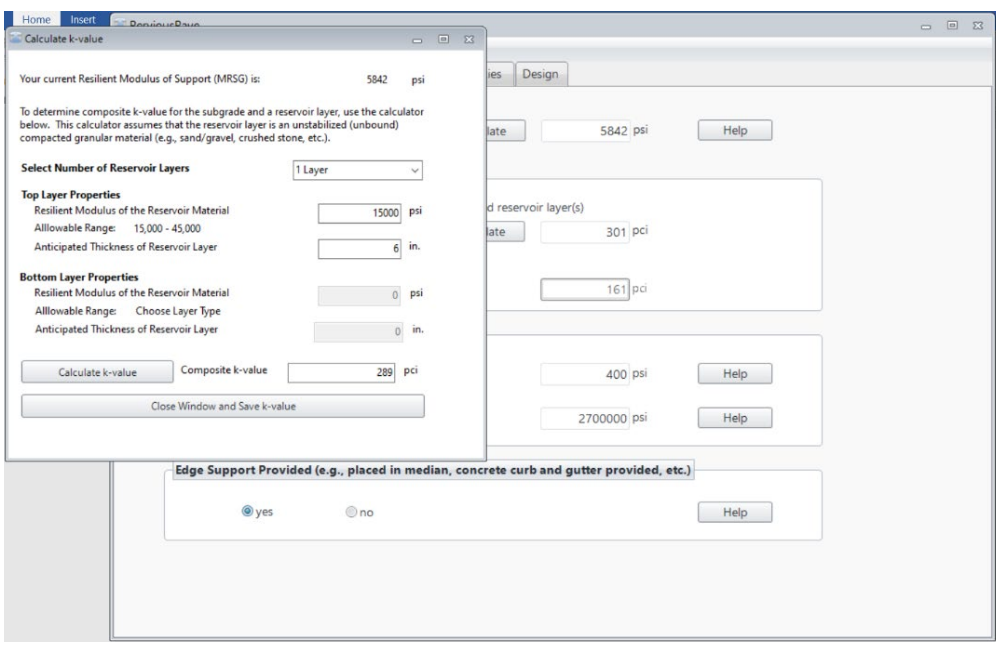
APPENDIX C: PerviousPave Results from Sensitivity Analysis

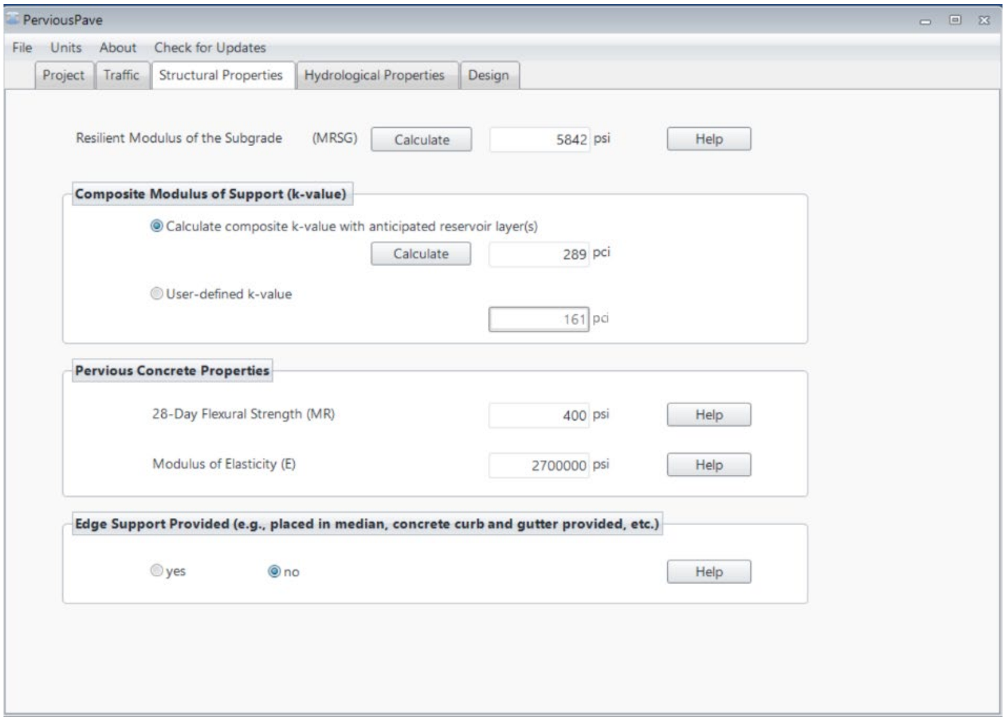
The screenshots below show a design example from PerviousPave. The final results of the scenarios from PerviousPave are also shown below.

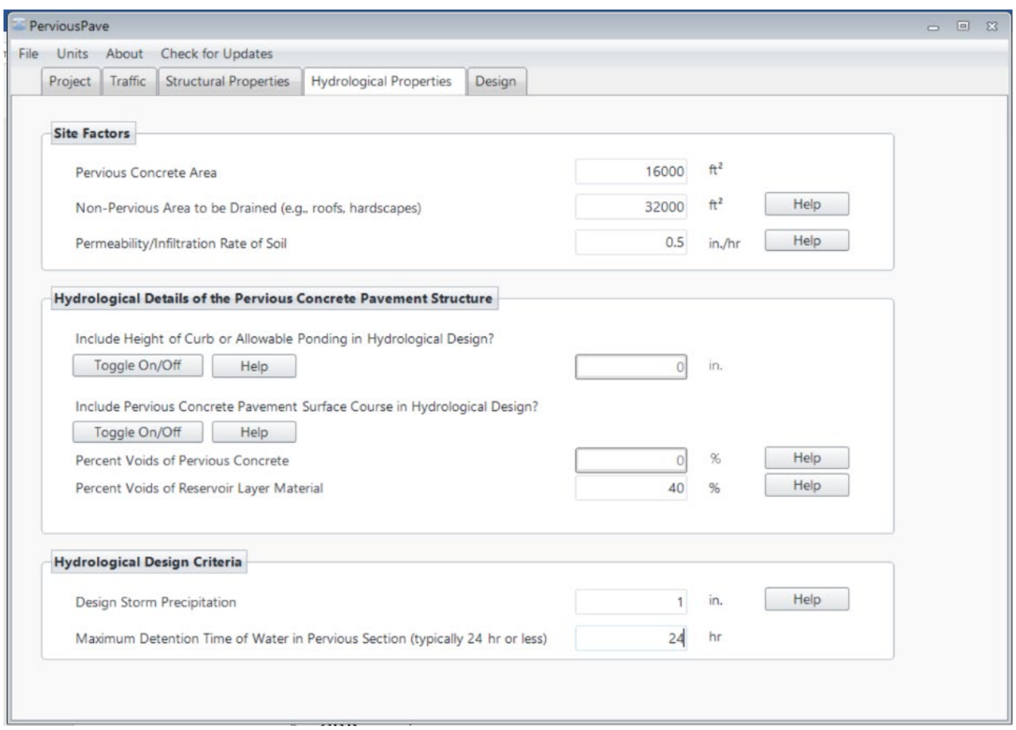


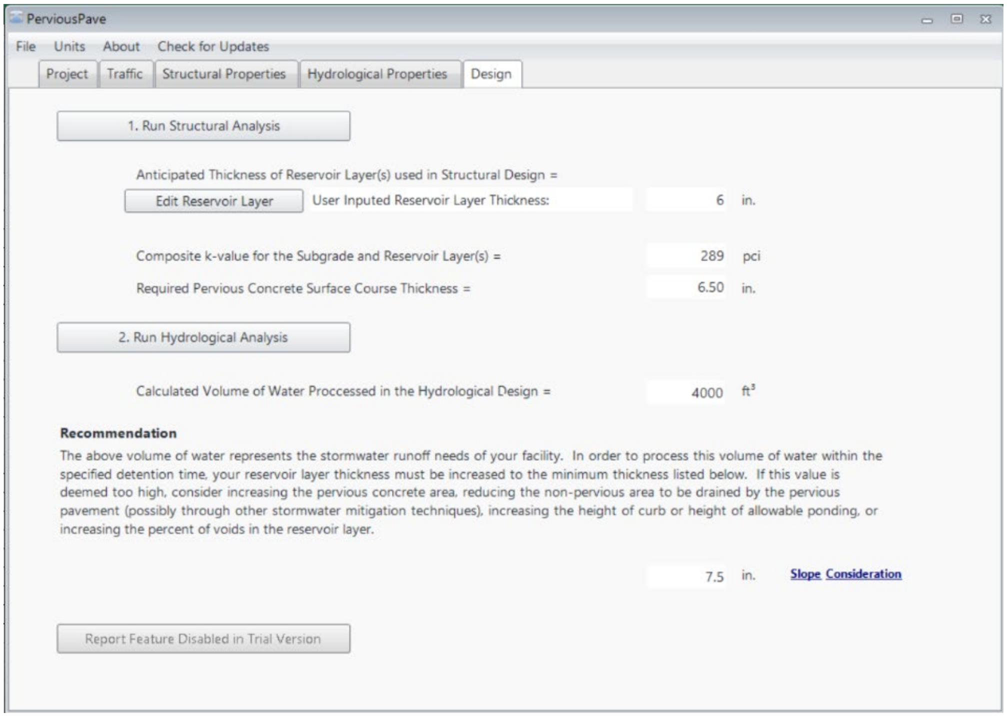


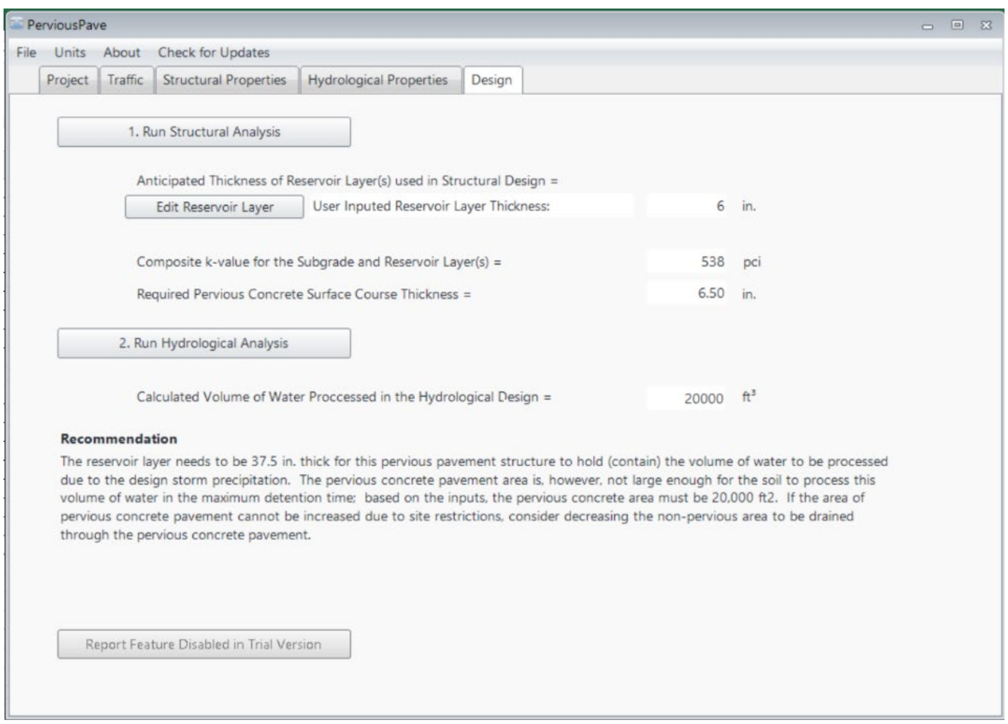

Calculate MRSG
You may either directly enter the MRSG value below, or use correlations to California Bearing Ratio (CBR) or Resistance Value (R-value) to estimate MRSG. PerviousPave will calculate MRSG using your input.
MRSG
California Bearing Ratio (CBR)
5
Resistance Value (R-value)
10.625
M RSG
5842
psi
Save MRSG and Close Window


Calculate k-value
Your current Resilient Modulus of Support (MRSG) is:
5842
psi
To determine composite k-value for the subgrade and a reservoir layer, use the calculator below. This calculator assumes that the reservoir layer is an unstabilized (unbound) compacted granular material (e.g., sand/gravel, crushed stone, etc.).
Select Number of Reservoir Layers
1 Layer
Top Layer Properties
Resilient Modulus of the Reservoir Material
15000
psi
Allowable Range: 15,000 - 45,000
Anticipated Thickness of Reservoir Layer
6
in.
Bottom Layer Properties
Resilient Modulus of the Reservoir Material
0
psi
Allowable Range: Choose Layer Type
Anticipated Thickness of Reservoir Layer
0
in.
Calculate k-value
Composite k-value
289
pci
Close Window and Save k-value
Design
5842 psi
Help
301 pci
161 pci
400 psi
Help
2700000 psi
Help
Edge Support Provided (e.g., placed in median, concrete curb and gutter provided, etc.)
yes
no
Help


PerviousPave
File Units About Check for Updates
Project Traffic Structural Properties Hydrological Properties Design
Resilient Modulus of the Subgrade (MRSG) Calculate 5842 psi Help
Composite Modulus of Support (k-value)
Calculate composite k-value with anticipated reservoir layer(s)
Calculate 289 pci
User-defined k-value
161 pci
Pervious Concrete Properties
28-Day Flexural Strength (MR) 400 psi Help
Modulus of Elasticity (E) 2700000 psi Help
Edge Support Provided (e.g., placed in median, concrete curb and gutter provided, etc.)
yes no Help


PerviousPave
File Units About Check for Updates
Project Traffic Structural Properties Hydrological Properties Design
Site Factors
Pervious Concrete Area 16000 ft²
Non-Pervious Area to be Drained (e.g., roofs, hardscapes) 32000 ft² Help
Permeability/Infiltration Rate of Soil 0.5 in./hr Help
Hydrological Details of the Pervious Concrete Pavement Structure
Include Height of Curb or Allowable Ponding in Hydrological Design?
Toggle On/Off Help 0 in.
Include Pervious Concrete Pavement Surface Course in Hydrological Design?
Toggle On/Off Help
Percent Voids of Pervious Concrete 0 % Help
Percent Voids of Reservoir Layer Material 40 % Help
Hydrological Design Criteria
Design Storm Precipitation 1 in. Help
Maximum Detention Time of Water in Pervious Section (typically 24 hr or less) 24 hr


PerviousPave
File Units About Check for Updates
Project Traffic Structural Properties Hydrological Properties Design
1. Run Structural Analysis
Anticipated Thickness of Reservoir Layer(s) used in Structural Design =
Edit Reservoir Layer
User Inputed Reservoir Layer Thickness:
6 in.
Composite k-value for the Subgrade and Reservoir Layer(s) =
289 pci
Required Pervious Concrete Surface Course Thickness =
6.50 in.
2. Run Hydrological Analysis
Calculated Volume of Water Proccessed in the Hydrological Design =
4000 ft³
Recommendation
The above volume of water represents the stormwater runoff needs of your facility. In order to process this volume of water within the specified detention time, your reservoir layer thickness must be increased to the minimum thickness listed below. If this value is deemed too high, consider increasing the pervious concrete area, reducing the non-pervious area to be drained by the pervious pavement (possibly through other stormwater mitigation techniques), increasing the height of curb or height of allowable ponding, or increasing the percent of voids in the reservoir layer.
7.5 in.
Slope Consideration
Report Feature Disabled in Trial Version


PerviousPave
File Units About Check for Updates
Project Traffic Structural Properties Hydrological Properties Design
1. Run Structural Analysis
Anticipated Thickness of Reservoir Layer(s) used in Structural Design =
Edit Reservoir Layer
User Inputed Reservoir Layer Thickness:
6 in.
Composite k-value for the Subgrade and Reservoir Layer(s) =
538 pci
Required Pervious Concrete Surface Course Thickness =
6.50 in.
2. Run Hydrological Analysis
Calculated Volume of Water Proccessed in the Hydrological Design =
20000 ft³
Recommendation
The reservoir layer needs to be 37.5 in. thick for this pervious pavement structure to hold (contain) the volume of water to be processed due to the design storm precipitation. The pervious concrete pavement area is, however, not large enough for the soil to process this volume of water in the maximum detention time; based on the inputs, the pervious concrete area must be 20,000 ft2. If the area of pervious concrete pavement cannot be increased due to site restrictions, consider decreasing the non-pervious area to be drained through the pervious concrete pavement.
Report Feature Disabled in Trial Version

Scenario 1:

AADTT	CBR (%)							
	4	5	6	7	8	9	10	15
2	6.5	6.5	6.5	6	6	6	6	6
3	6.5	6.5	6.5	6.5	6	6	6	6
4	6.5	6.5	6.5	6.5	6.5	6.5	6	6
5	6.5	6.5	6.5	6.5	6.5	6.5	6.5	6
6	7	6.5	6.5	6.5	6.5	6.5	6.5	6
7	7	6.5	6.5	6.5	6.5	6.5	6.5	6
8	7	7	6.5	6.5	6.5	6.5	6.5	6
9	7	7	6.5	6.5	6.5	6.5	6.5	6.5
10	7	7	7	6.5	6.5	6.5	6.5	6.5

Scenario 2:

Precipitation (in.)	Infiltration Rate of Subgrade (in./hr)						
	0.5	0.75	1	1.25	1.5	1.75	2
1	7.5	7.5	7.5	7.5	7.5	7.5	7.5
1.2	9	9	9	9	9	9	9
1.25	9.38	9.38	9.38	9.38	9.38	9.38	9.38
1.5	11.25	11.25	11.25	11.25	11.25	11.25	11.25
2	15	15	15	15	15	15	15
2.25	16.88	16.88	16.88	16.88	16.88	16.88	16.88
2.5	18.75	18.75	18.75	18.75	18.75	18.75	18.75
2.75	20.62	20.62	20.62	20.62	20.62	20.62	20.62
3	22.5	22.5	22.5	22.5	22.5	22.5	22.5
3.25	24.38	24.38	24.38	24.38	24.38	24.38	24.38
3.5	26.25	26.25	26.25	26.25	26.25	26.25	26.25
3.75	28.12	28.12	28.12	28.12	28.12	28.12	28.12
4	30	30	30	30	30	30	30
4.25	31.88	31.88	31.88	31.88	31.88	31.88	31.88
4.5	33.75	33.75	33.75	33.75	33.75	33.75	33.75
4.75	35.62	35.62	35.62	35.62	35.62	35.62	35.62
5	37.5	37.5	37.5	37.5	37.5	37.5	37.5
5.25	39.38	39.38	39.38	39.38	39.38	39.38	39.38
5.5	41.25	41.25	41.25	41.25	41.25	41.25	41.25
5.75	43.12	43.12	43.12	43.12	43.12	43.12	43.12
6	45	45	45	45	45	45	45

APPENDIX D: Results from NAPA Guide – Sensitivity Analysis

Scenario 1:

Table B-1. Required porous asphalt layer thickness for W_{18} = 27,000 ESALs (Traffic Class II, 20-year design life in Table 3).

		Design Subgrade Resilient Modulus (psi)						
		2000	3000	4000	6000	8000	10,000	12,000
Base Thickness (in)	6	5.5	4.5	4.5	4	3.5	3.5	3
	12	4.5	4	3.5	3.5	3	3	3
	18	4	4	3.5	3	3	3	3
	24	3.5	3.5	3.5	3	3	3	3
	30	3.5	3.5	3	3	3	3	3
	36	3.5	3.5	3	3	3	3	3
	42	3.5	3.5	3	3	3	3	3
	48	3.5	3.5	3	3	3	3	3

Scenario 2:

Table B-2. Required porous asphalt layer thickness for W_{18} = 110,000 ESALs (Traffic Class III, 20-year design life in Table 3).

		Design Subgrade Resilient Modulus (psi)						
		2000	3000	4000	6000	8000	10,000	12,000
Base Thickness (in)	6	7	6	5.5	5	4.5	4.5	4
	12	5.5	5	5	4.5	4	4	4
	18	5	5	4.5	4.5	4	4	4
	24	5	4.5	4.5	4	4	4	4
	30	4.5	4.5	4	4	4	4	4
	36	4.5	4.5	4	4	4	4	4
	42	4.5	4	4	4	4	4	3.5
	48	4	4	4	4	4	3.5	3.5

Scenario 3:

Table B-3. Required porous asphalt layer thickness for W_{18} = 820,000 ESALs (Traffic Class IV, 15-year design life in Table 3).

		Design Subgrade Resilient Modulus (psi)						
		2000	3000	4000	6000	8000	10,000	12,000
Base Thickness (in)	6	9.5	8.5	7.5	7	6.5	6	6
	12	8	7	6.5	6	6	5.5	5.5
	18	7	6.5	6.5	6	5.5	5.5	5.5
	24	6.5	6.5	6	6	5.5	5.5	5.5
	30	6.5	6	6	5.5	5.5	5.5	5.5
	36	6	6	5.5	5.5	5.5	5.5	5.5
	42	6	6	5.5	5.5	5.5	5.5	5
	48	6	5.5	5.5	5.5	5.5	5	5

APPENDIX E: Results from ICPI Guide – Sensitivity Analysis

PEDESTRIAN								
	Soaked CBR (R-value)	4 (9)	5 (11)	6 (12.5)	7 (14)	8 (15.5)	9 (17)	10 (18)
	Resilient Modulus, MPa*	43	49	55	61	67	72	77
	Base thickness, mm ASTM No. 57	150	150	150	150	150	150	150

VEHICULAR								
	Soaked CBR (R-value)	4 (9)	5 (11)	6 (12.5)	7 (14)	8 (15.5)	9 (17)	10 (18)
	Resilient Modulus, MPa*	43	49	55	61	67	72	77
Lifetime ESALs (Traffic Index)								
50,000 (6.3) and Residential Driveways	Base thickness, mm ASTM No. 57	100	100	100	100	100	100	100
	Subbase thickness mm ASTM No. 2	150	150	150	150	150	150	150
100,000 (6.8)	Base thickness, mm ASTM No. 57	100	100	100	100	100	100	100
	Subbase thickness mm ASTM No. 2	200	150	150	150	150	150	150
200,000 (7.4)	Base thickness, mm ASTM No. 57	100	100	100	100	100	100	100
	Subbase thickness mm ASTM No. 2	325	275	225	175	150	150	150
300,000 (7.8)	Base thickness, mm ASTM No. 57	100	100	100	100	100	100	100
	Subbase thickness mm ASTM No. 2	400	350	300	250	225	200	175
400,000 (8.1)	Base thickness, mm ASTM No. 57	100	100	100	100	100	100	100
	Subbase thickness mm ASTM No. 2	475	400	350	300	275	250	225
500,000 (8.3)	Base thickness, mm ASTM No. 57	100	100	100	100	100	100	100
	Subbase thickness mm ASTM No. 2	525	450	400	350	300	275	250
600,000 (8.5)	Base thickness, mm ASTM No. 57	100	100	100	100	100	100	100
	Subbase thickness mm ASTM No. 2	550	475	425	375	350	300	275
700,000 (8.6)	Base thickness, mm ASTM No. 57	100	100	100	100	100	100	100
	Subbase thickness mm ASTM No. 2	600	525	450	425	375	350	300
800,000 (8.8)	Base thickness, mm ASTM No. 57	100	100	100	100	100	100	100
	Subbase thickness mm ASTM No. 2	625	550	500	450	400	375	325
900,000 (8.9)	Base thickness, mm ASTM No. 57	100	100	100	100	100	100	100
	Subbase thickness mm ASTM No. 2	650	575	525	475	425	400	350
1,000,000 (9)	Base thickness, mm ASTM No. 57	100	100	100	100	100	100	100
	Subbase thickness mm ASTM No. 2	675	600	525	475	425	400	375

*M_r in psi = 2,555 x $CBR^{0.64}$; M_r in MPa = 17.61 x $CBR^{0.64}$

Assumptions: 80% confidence level

Commercial vehicles = 10%; Average ESALs per commercial vehicle = 2

ASTM No. 57 stone layer coefficient = 0.09; ASTM No. 2 stone layer coefficient = 0.06

ASTM No. 3 or 4 stone may be substituted for ASTM No. 2 stone subbase layer.

80 mm thick concrete pavers and 50 mm ASTM No. 8 bedding layer coefficient = 0.3

Total PICP cross section depth equals the sum of the subbase, base, 50 mm bedding and paver 80 mm thickness.

Consult with geogrid manufacturers for base/subbase thickness recommendations using geogrids.

APPENDIX F: Case Study

- **Cost calculations of parking lot components:**

BASE BID

SCHEDULE OF ESTIMATED QUANTITIES AND BID PRICES

ITEM	DESCRIPTION	UNIT	QUANTITY	COST	TOTAL COST
1	Bioretention	LS	1	$31,860.00	$31,860.00
2	Brick Pavers w/ Detention	SQFT	7800	$19.40	$151,320.00
3	Clearing and Grubbing, (approx. 1 acre)	LS	1	$5,000.00	$5,000.00
4	Concrete Retaining Wall	SQFT	2,450	$64.25	$157,412.50
5	Concrete, 6" Thick	SQYD	125	$75.00	$9,375.00
6	Concrete Steps w/ Handrail	LF	20	$425.00	$8,500.00
7	Crushed Aggregate Base, 6" Layer	SQYD	3,250	$7.25	$23,562.50
8	Curb & Gutter	LF	1,400	$10.00	$14,000.00
37	Superpave Bit. Concrete Wearing Layer, Leveling, , 3/8" Max Agg	TON	15	$85.00	$1,275.00
38	Superpave Bit. Concrete Wearing Layer, 3/8" Max Agg	TON	180	$85.00	$15,300.00
39	Superpave Bit. Concrete Binder Layer, 3/4" Max Agg	TON	360	$85.00	$30,600.00

Total Price = $305,917.5

- Bioretention unit cost:
  - The pond has an obelisk shape and its $Volume = \frac{h}{6}(Ab + aB + 2(ab + AB))$

    $Volume = \frac{3.5ft}{6}\big(154ft \times 21ft + 140ft \times 36ft + 2(140ft \times 21ft + 154ft \times 36)\big) = 14{,}724.5\ ft^3$. Since the tool gives the dimensions of the soil media and the pond combined, the empty volume of the pond was converted to a volume with 40% void ratio like in the tool: $Volume = \frac{14{,}724.5\ ft^3}{0.40} = 36{,}811.25ft^3$
  - The soil media is rectangular and its $Volume = 140ft \times 21ft \times 4.5ft = 13{,}230\ ft^3$
  - $Unit\ cost = \frac{\$31{,}860}{36{,}811.25ft^3 + 13{,}230\ ft^3} = 0.64\frac{\$}{ft^3}$
- Crushed Aggregate Base unit cost:
  - The area of HMA is 3,250 yd^2, which is equivalent of 29,250 ft^2

- $Unit\ cost = 7.25\frac{\$}{yd^2} = 0.805\frac{\$}{ft^2}$ and $1.61\frac{\$}{ft^3}$ since the base is 6" thick:

- PICP unit cost:
  - The expression "w/ detention" was used to represent the crushed aggregate base/subbase. Therefore, the price of the base/subbase was first removed from the unit cost provided. The area of PICP is 7,800 ft^2 and base/subbase is 30" thick:

    $\left(7{,}800ft^2 \times \frac{30"}{12"/'}\right) \times \frac{\$1.61}{ft^3} = \$31{,}195 \rightarrow \$151{,}320 - \$31{,}195 = \$119{,}925$

  - $Unit\ cost = \frac{\$119{,}925}{7{,}800ft^2} = 15.375\frac{\$}{ft^2}$

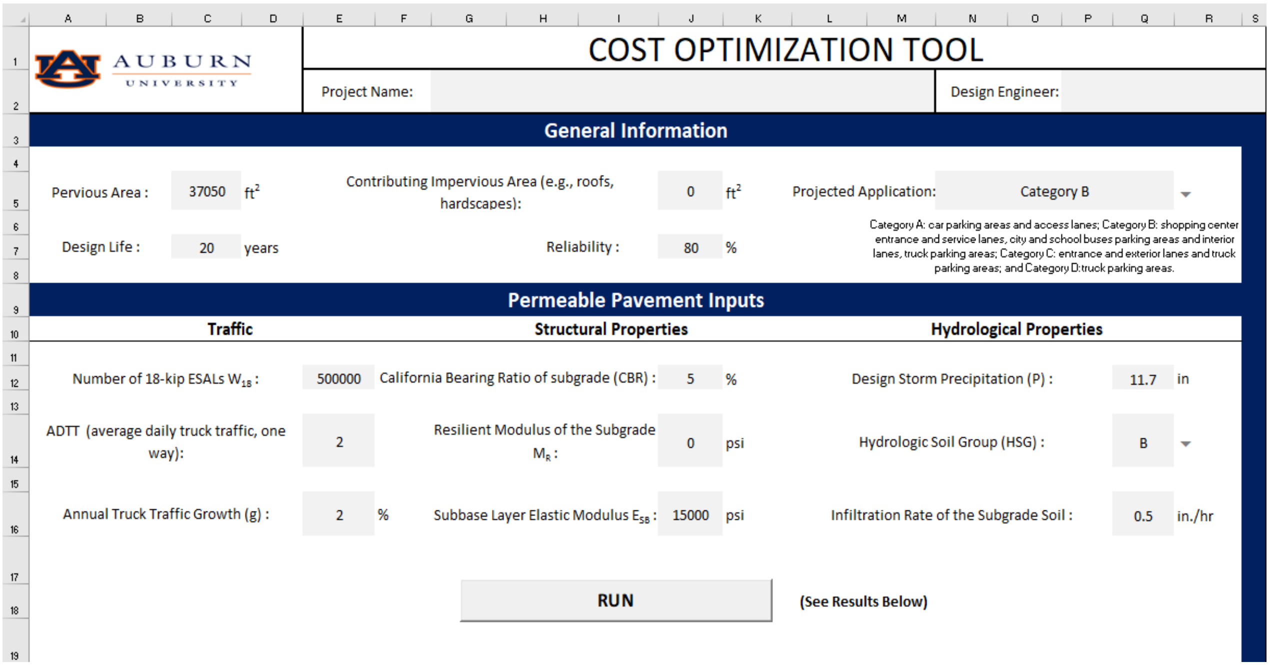
- HMA unit cost:
  - The total price was given as $\$1{,}275 + \$15{,}300 + \$30{,}600 = \$47{,}175$ and the area of HMA is 29,250 ft^2: $\frac{\$47{,}175}{29{,}250ft^2} = 1.613\frac{\$}{ft^2}$

- Excavation unit cost:
  - The excavation includes the entire PICP section that is 33.125" $\rightarrow volume =$ $\left(7{,}800ft^2 \times \frac{33.125"}{12"/'}\right) = 21{,}531.25ft^3$ and Bioretention $\rightarrow volume =$ $36{,}811.25ft^3 + 13{,}230\ ft^3 = 50{,}041.25ft^3$.
  - $Unit\ cost = \frac{\$52{,}000}{21{,}531.25ft^3 + 50{,}041.25ft^3} = 0.73\frac{\$}{ft^3}$

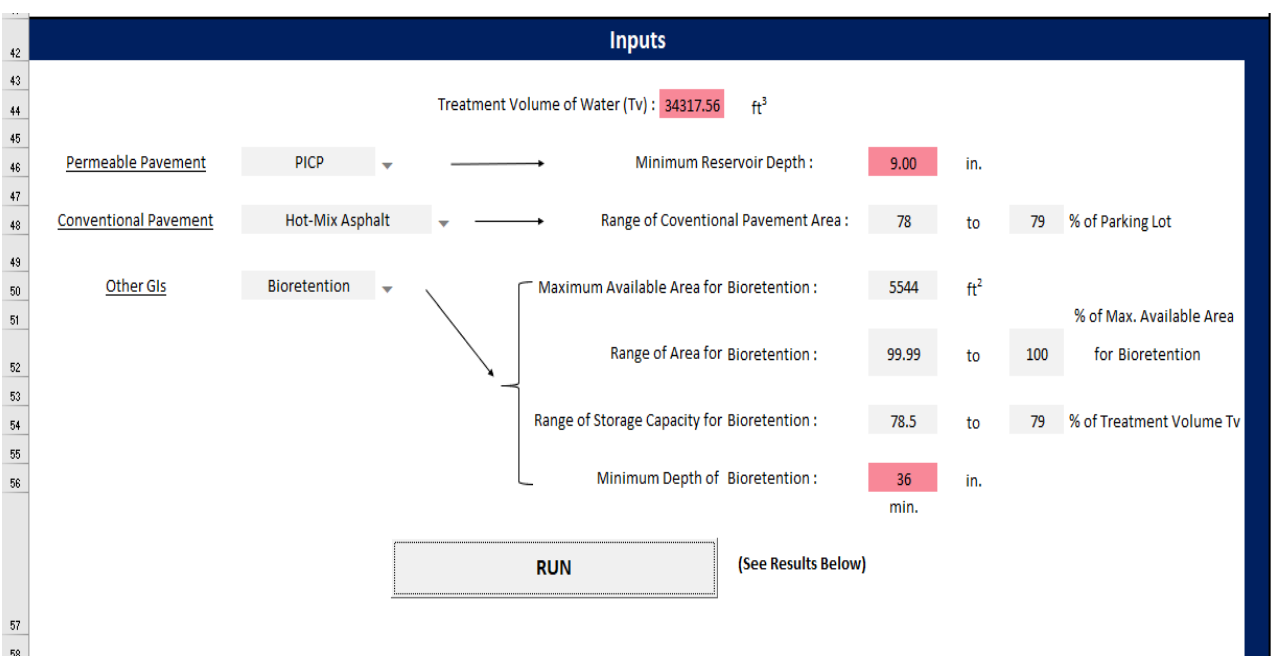
The unit costs calculated above were used for the case study by entering the value in the tool.

Scenarios

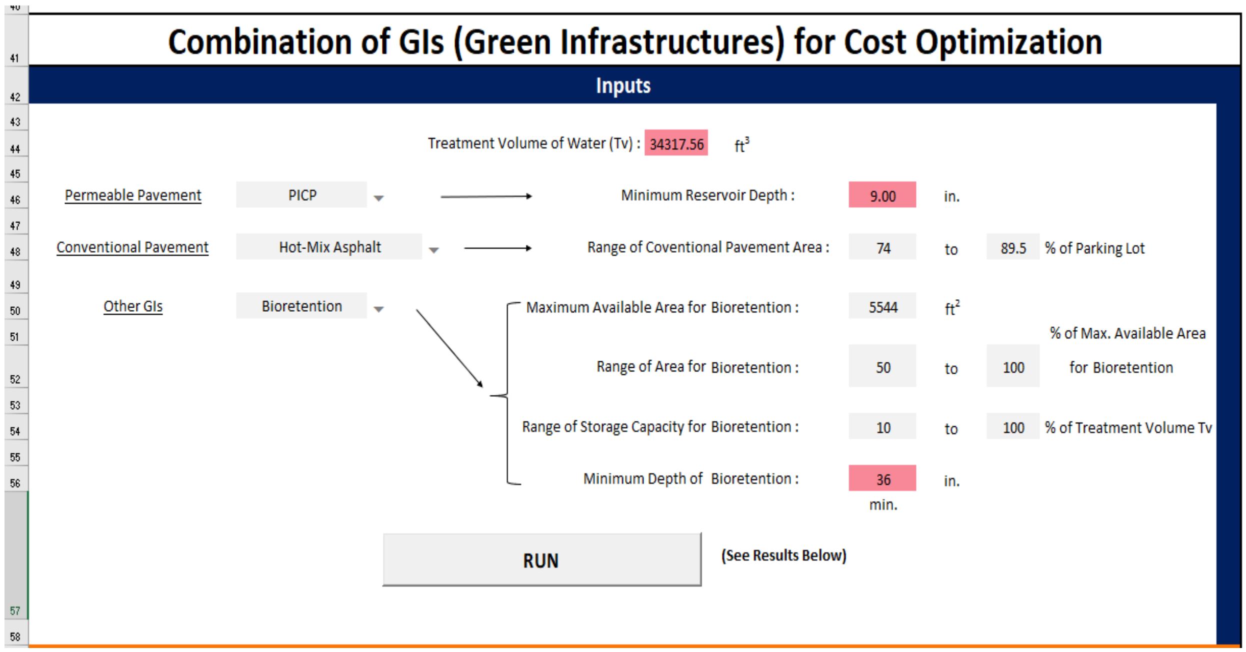
General Inputs:



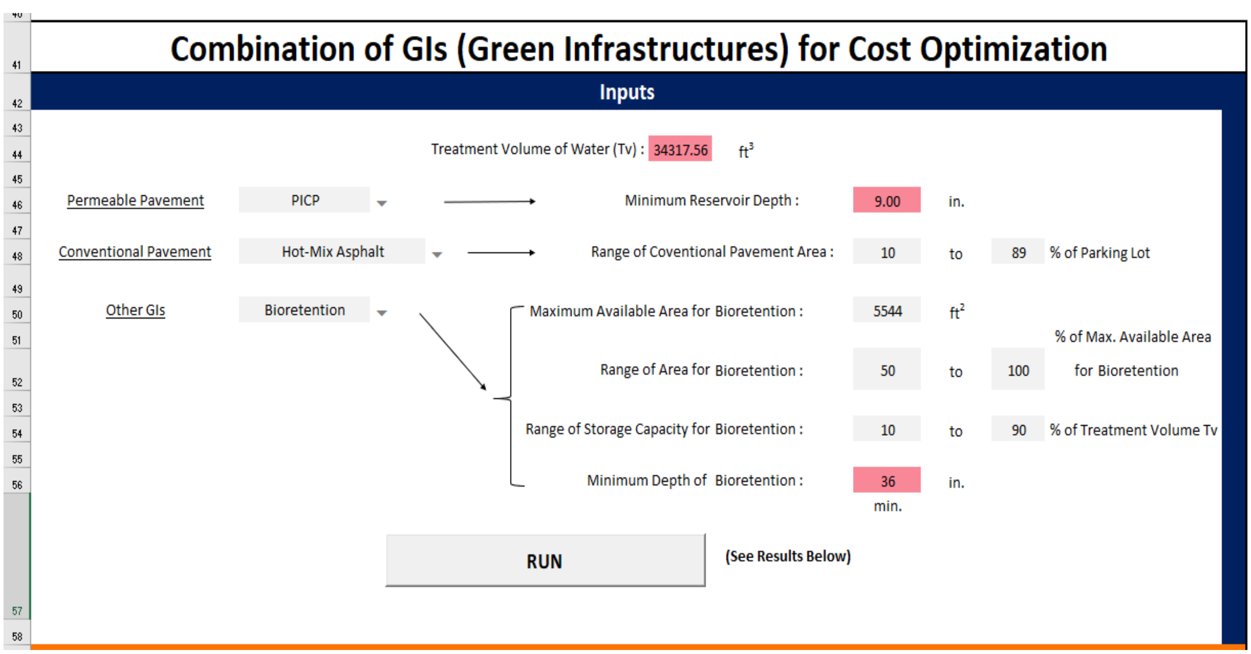
Reference optimization inputs to match original existing cross section:

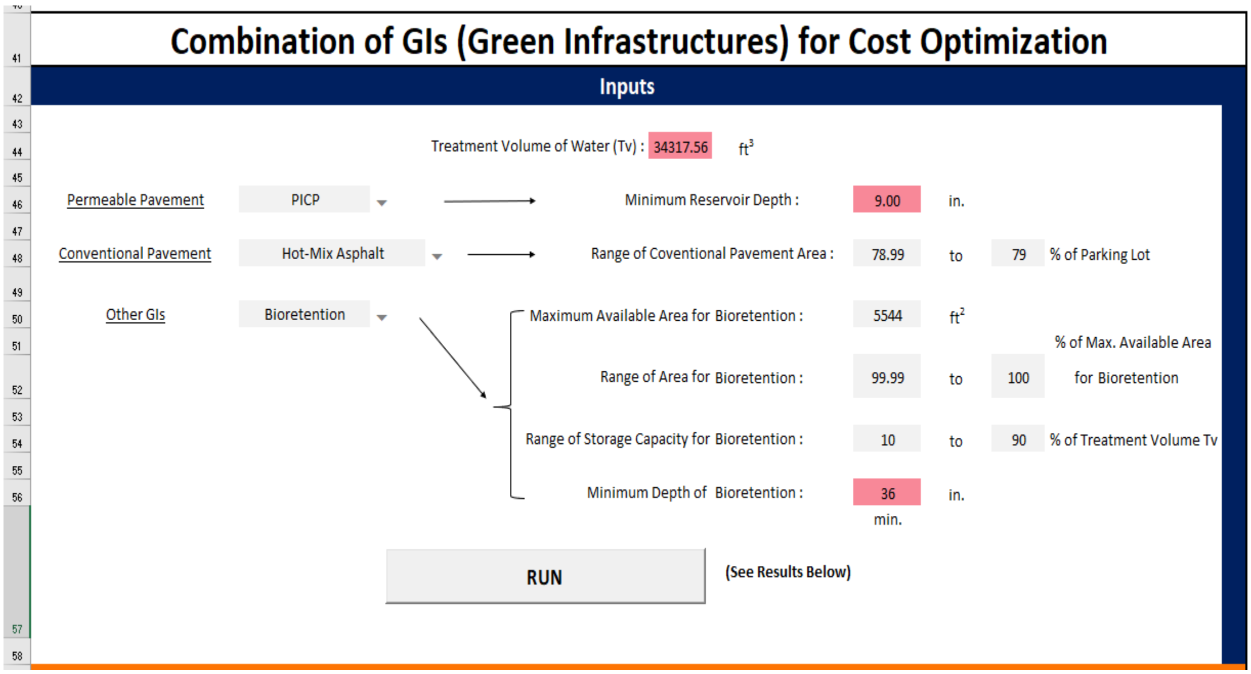


Scenario 1:



Scenario 2:





Combination of GIs (Green Infrastructures) for Cost Optimization

Inputs

Treatment Volume of Water (Tv) : 34317.56 ft^3

Permeable Pavement	PICP	Minimum Reservoir Depth :	9.00	in.		
Conventional Pavement	Hot-Mix Asphalt	Range of Coventional Pavement Area :	78.99	to	79	% of Parking Lot
Other GIs	Bioretention	Maximum Available Area for Bioretention :	5544	ft^2		
		Range of Area for Bioretention :	99.99	to	100	% of Max. Available Area for Bioretention
		Range of Storage Capacity for Bioretention :	10	to	90	% of Treatment Volume Tv
		Minimum Depth of Bioretention :	36 min.	in.		

RUN **(See Results Below)**