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Permeable Interlocking Concrete Pavement

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Preface

Permeable pavements typically consist of pervious concrete, porous asphalt, or interlocking concrete paver units over an open-graded base or subbase layer(s). Permeable pavements are designed to infiltrate stormwater, reduce peak flows, improve stormwater quality, and promote groundwater recharge. They have become an integral part of low-impact development, sustainable design, green infrastructure, and best management practices for stormwater management. In order to be effective within municipal road networks, permeable pavements must be designed to provide sufficient structural capacity to accommodate the anticipated vehicle loadings while managing stormwater flows into and out of the permeable pavement.

Although there have been many well designed and constructed permeable pavements, this is a relatively new technology compared to conventional pavements, and there have been some performance concerns. These concerns include pavement surface rutting caused by loads exceeding the pavement structural design, settlement caused by improper base and/or subbase gradations, and insufficient compaction of the base or subbase. A common concern is clogging of the pavement surface from sediments tracked onto the pavement or transported by water run-on from adjacent surfaces.

This standard was written to address these concerns and more. It provides design, construction, and maintenance guidance for permeable interlocking concrete pavement to achieve stormwater management goals while providing a structurally adequate pavement section to accommodate the anticipated vehicular loading in a cost-efficient manner.

Introduction

Permeable interlocking concrete pavement (PICP) can provide a durable and effective pavement and stormwater management system. As with any pavement and stormwater management practice, proper design, construction, and maintenance procedures are required. To better address these needs, this standard was prepared by the ASCE Permeable Interlocking Concrete Pavement Committee. This publication establishes guidelines for developing appropriate pavement structures for various stormwater drainage, traffic, and subgrade conditions as well as providing guidance on construction and maintenance.

This standard is written with the intent of being adopted in whole or in part for use by national, provincial, state, and local stormwater and road agencies for the consistent and effective design, construction, and maintenance of permeable interlocking concrete pavement systems. The overall goal is assisting design professionals, civil engineers, the industry, public stormwater and transportation agencies, and the wider public by establishing design standards for permeable interlocking concrete pavements. The document provides

- Definitions of terms common to permeable pavements;
- Methods for structural design to accommodate incidental and frequent vehicular use;
- Methods for hydrologic design to accommodate water infiltration and flow into, within, and out of the pavement system;
- Construction and inspection procedures;
- Guide construction specifications; and
- Maintenance procedures for the permeable pavement system.

PICP may help achieve compliance with many national, provincial, state, and local regulations, as well as transportation agency design requirements for the control of stormwater runoff. Requirements may include the following:

- Compliance with federal, provincial, state, and local transportation design standards;
- Compliance with pavement structural design and construction requirements;
- Compliance with vehicular and pedestrian safety and access requirements;
- Transportation asset management compliance, including lifecycle cost analysis and lifecycle assessment of environmental impacts;
- Stormwater runoff controls and regulatory compliance;
- Compliance with groundwater protection requirements;
- Postconstruction runoff volume and pollutant control for new development and redevelopment;
- Reductions in impervious cover (i.e., roofs and pavements) and resulting runoff;
- Runoff volume storage and/or infiltration to reduce overflows, especially combined sewer overflows, as well as reduction of flooding for a more resilient infrastructure;

- Compliance with total maximum daily load (TMDL) requirements for receiving waters;
- Management of water quality and/or quantity storm events; and
- Compliance with local building code requirements.

Permeable interlocking concrete pavements may assist in achieving regulatory program and policy compliance. Examples include the Great Lakes Protection Act, Species at Risk Act, National Pollutant Discharge Elimination System (NPDES), Canadian Federal Fisheries Act, U.S. Clean Water Act, U.S. Environmental Protection Agency (US EPA) Stormwater Assessment Program, Source Water Protection Plans, CALGreen in California, the International Green Construction Code, ASHRAE Standard 189.1, and stormwater utility fee credits or other codes that require compliance with Leadership in Energy and Environmental Design (LEED) or similar sustainable design and construction rating systems.

Nonregulatory drivers that influence PICP use include the following: economics that often make PICP a lower cost alternative to conventional drainage and stormwater management system designs, and project owner preference for conformance to sustainable rating systems for roads and transportation infrastructure. Examples include the Green Business Certification, Inc.'s Sustainable SITES Initiative, the Institute for Sustainable Infrastructure's Envision evaluation system, Greenroads, GreenPave, and the Federal Highway Administration's Infrastructure Voluntary Evaluation Sustainability Tool (INVEST).

Finally, other nonregulatory drivers include product, system, or project lifecycle analysis (LCA) of environmental impacts in the manufacture, construction, use, and end-of-life phase. Product Category Rules (PCRs) for segmental concrete paving products used in PICP are available from ASTM. These rules provide a useful framework for conducting an LCA, as well as for providing environmental product declarations for paving products.

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Acknowledgments

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Chapter 1

GENERAL

1.1 SCOPE

This standard establishes hydrologic and structural design methods for permeable interlocking concrete pavement (PICP). Permeable pavement design is typically site specific and requires careful consideration of structural and hydrologic conditions, and of the impact on the surrounding environment.

This document provides information for professionals to use in the design of permeable pavement systems. This information includes applicable standards, definitions, best practices, structural and hydrologic design, key design elements, guide specifications, construction guidelines, and long-term maintenance. This standard is recommended for roadways with design speeds no greater than 50 kph (35 mph) receiving less than 1 million equivalent single axle loads (ESALs).

The pavement structural design recommendations in this standard are based on mechanistic-empirical design research. The process described herein is based on data from a full-scale accelerated PICP load testing program. Design considerations herein require a working knowledge of stormwater drainage, soil mechanics, traffic loading, and pavement materials. Pavement design and stormwater specialists should be consulted for the application of this standard.

This book consists of six chapters plus appendixes and references:

- Chapter 1 outlines the general scope of the standard;
- Chapter 2 provides definitions of key terms, as well as a list of consensus standards and other referenced documents used in this standard;
- Chapter 3 provides preliminary information required to design permeable pavements and also provides fundamental information on typical designs for common site conditions;
- Chapter 4 provides detailed procedures and rationale for structural and hydrologic design;
- Chapter 5 covers construction and inspection guidelines; and
- Chapter 6 includes pavement maintenance guidelines.

The appendixes are the following:

- Appendix A: Design Examples,
- Appendix B: Guide Construction Specifications,
- Appendix C: Examples of Orifice and Common Weir Equations,

- Appendix D: PICP Structural Design Using AASHTO 1993, *Guide for Design of Pavement Structures*, and
- Appendix E: Approximate Correlation between Permeability and Unified Soil Classification.

This standard applies to the design of new and retrofit pavement systems surfaced with PICP. It also includes hydrologic and structural design guidelines using open-graded aggregate bases. This standard does not address the specific biological, chemical, and physical processes as related to water quality improvement. Other references should be consulted for the water quality benefits available to project-specific pavement systems.

This standard does not address site-specific constraints and/or local conditions that may require more detailed analysis, specifications, construction, and maintenance practices critical to successful pavement performance.

1.2 REFERENCED STANDARDS

In addition to provincial, state, and local government jurisdiction over stormwater drainage and pavement structural design procedures, the provisions of applicable ASTM, CSA, or AASHTO standards listed herein are provided in full in the list of consensus standards and are part of this standard. Many ASTM standards and test methods have equivalent standards and test methods in AASHTO *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*. Equivalent standards published by both organizations are used throughout this standard.

1.3 VARIATIONS FROM THIS STANDARD

Use of proprietary, new and/or improved permeable interlocking concrete pavers, aggregate materials, evaluation techniques, and installation methods are not specifically excluded, as long as the design and installation of the pavement are shown to comply with or exceed this standard.

1.4 ENGINEER REQUIRED

Work covered by this standard should be carried out under the guidance of a professional engineer with a background in the design of pavement and stormwater systems. The professional engineer is hereinafter referred to as the engineer. This standard does not dictate the means and methods to be used by the engineer. Means and methods must be appropriate to each project.

Chapter 2
DEFINITIONS

2.1 GENERAL

This chapter defines specific terms common to permeable interlocking concrete pavements (PICPs) used in this standard. *ICPI Tech Spec 1: Glossary of Terms for Segmental Concrete Pavement* (1995, 2017) can be referenced for defining additional terms used by the interlocking concrete pavement industry.

Terms not defined in this standard should use their ordinarily accepted meaning within the context in which they are written. *Webster's New International Dictionary of the English Language, Unabridged*, latest edition, https://www.merriam-webster.com, should be considered as providing ordinarily accepted meanings.

2.2 TERMS

AASHTO: American Association of State Highway and Transportation Officials.

Adsorption: The adhesion of pollutants commonly found in stormwater runoff to the PICP materials and soil subgrade to improve water quality.

Aggregate: Crushed stone used for jointing, bedding, base, and subbase materials.

Angularity: The sharpness of edges and corners of particles. Used to describe sand, stone, and aggregates.

Aquifer: A porous, water-bearing geologic formation that yields water for human and environmental use.

Aspect Ratio: The longest overall length of a paver divided by its thickness. Example: A paver that is 100 mm (4 in.) wide by 200 mm (8 in.) long by 80 mm (3 1/8 in.) thick has an aspect ratio of 2.5.

ASTM: American Society for Testing and Materials International.

ASTM No. 2 Stone: As defined by ASTM D448, the CSA A23.1 approximation would be Group II 80-40.

ASTM No. 8 Stone: As defined by ASTM D448, the CSA A23.1 approximation would be Group II 5-2.5.

ASTM No. 57 Stone: As defined by ASTM D448, the CSA A23.1 approximation would be Group II 28-14.

Base or Base Course: A material of a designed thickness placed under the surface wearing course of paving units and its bedding course. It is placed over a subbase or a subgrade to support the surface course and bedding materials. A base course can be compacted aggregate, cement or asphalt stabilized aggregate, asphalt, or concrete. This material generally conforms to the grading requirements of ASTM No. 57 stone and is generally 100 mm (4 in.) thick. Sometimes called a choker course. All layers within a permeable pavement are open graded.

Bedding Course: A layer of coarse crushed and washed stone screeded smooth as bedding for the pavers. This material generally conforms to the grading requirements of ASTM No. 8 stone. A screeded layer is generally 50 mm (2 in.) thick.

Best Management Practice (BMP): A structural (or nonstructural) measure designed to infiltrate, temporarily store, or treat stormwater runoff in order to reduce pollution and/or flooding. Also called a stormwater control measure or SCM.

Bioretention: A stormwater management practice that uses soils and vegetation to treat pollutants in urban runoff and to encourage infiltration of stormwater into the ground.

Bundle: Several layers (or clusters) of unit pavers stacked vertically, packaged, and tagged for shipment. Also sometimes called a cube.

California Bearing Ratio (CBR): A test method and result that renders an approximation (expressed as a percent) of the bearing strength of soil compared to that of a high-quality, compacted aggregate base. The test defines the ratio of (1) the force per unit area required to penetrate a soil mass with a 19 cm^2 (3 $in.^2$) circular piston (of approximately 51 mm (2 in.) diameter) at the rate of 1.3 mm/min (0.05 in./min), to (2) that force required for corresponding penetration of a standard material. The ratio is usually determined at 2.5 mm (0.1 in.) penetration, although other penetrations are sometimes used. *See* ASTM D1883 or AASHTO T-193 for more details.

Cation: A positively charged atom or group of atoms in soil particles that, through exchange with ions of metals in stormwater runoff, enable those metals to attach themselves to soil particles.

Chamfer: A 45° beveled edge around the top of a paver unit that is usually 2 to 6 mm (1/16 to 1/4 in.) wide. It allows water to drain from the paver surface, facilitates snow removal, helps prevent edge chipping, and delineates the individual unit pavers.

Choker course: A layer of aggregate placed or compacted into the surface of another layer to provide stability and a smoother surface. The particle sizes of the choker course are generally smaller than those of the surface into which it is being pressed but are not too small to pass through the underlying layer.

Clay Soils: 1. (Agronomy) Soils with particles less than 0.002 mm in size. 2. A soil textural class. 3. (Engineering). A fine-grained soil with more than 50% passing through the 0.075 mm (No. 200) sieve with a high plasticity index in relation to its liquid limit, according to the Unified Soil Classification System.

Cluster: The group of pavers forming a single layer from a bundle of pavers that can be held by the clamp of a unit paver laying machine during mechanical installation.

Coarse Aggregate: Aggregate predominantly retained on the 4.75 mm (No. 4) sieve; or that portion of an aggregate retained on the 4.75 mm (No. 4) sieve.

Combined Sewer System Overflow, or CSO: Conveyance of storm and sanitary sewage in the same pipes that, as a result of rain events, overflow and discharge into receiving waters. CSOs generally occur in older, more urban areas. CSOs do significant damage to water quality, resulting in diminished economic and recreational activities for receiving waters.

Compaction: The process of inducing close packing of solid particles such as soil, sand, or aggregate.

Compressive Strength: The measured maximum resistance of a concrete paver to loading expressed as force per unit cross-sectional area, such as newtons per square millimeter (megapascals) or pounds per square inch.

Concrete Block Pavement (CBP): *See* Interlocking Concrete Pavement (ICP).

Concrete Pavers: Precast concrete paver units meeting the requirements of ASTM C936 or CSA A231.2. For the purpose of this standard, they have an aspect ratio (length divided by thickness) of 3 or less.

Contributing Drainage Area (CDA) Ratio: The area contributing runoff into the PICP divided by the area of the PICP receiving that runoff.

Course: A row of pavers.

Crushed Stone: A product used for pavement bases made from mechanical crushing of rocks, boulders, or large cobblestones at a quarry. All faces of each aggregate have well-defined edges resulting from the crushing operation.

CSA: Canadian Standards Association.

CSA Group II 5-2.5 Stone: As defined by CSA 23.1. The ASTM D448 approximation is No. 8 Stone.

CSA Group II 28-14 Stone: As defined by CSA 23.1. The ASTM D448 approximation is No. 57 Stone.

CSA Group II 80-40 Stone: As defined by CSA 23.1. The ASTM D448 approximation is No. 2 Stone.

Cube: *See* Bundle.

Curve Number (CN): A numerical representation of a given area’s hydrologic soil group, plant cover, impervious cover, interception, and surface storage. The U.S. Soil Conservation Service (SCS) originally developed the concept. A curve number is used to help convert rainfall depth into runoff volume.

Dense-Graded Aggregate Base: A compacted crushed stone base whose gradation yields very small voids between the particles with no visible spaces between them. Most dense-graded bases have particles ranging in size from 38 mm (1 1/2 in.) or 19 mm (3/4 in.) down to fines passing the 0.075 mm (No. 200) sieve.

Density: The mass per unit volume.

Detention Pond or Structure: The temporary storage of stormwater runoff in an area with the objectives of decreasing peak discharge rates and providing a settling basin for pollutants.

Edge Restraint: A curb, edging, building, or other fixed object that surrounds the bedding course and pavers so that they do not spread and lose interlock under loading.

Equivalent Axle Loads (EALs): See Equivalent Single Axle Loads (ESALs).

Equivalent Single Axle Loads (ESALs): Summation of equivalent 80 kN (18,000 lb-force) single axle loads used to combine mixed traffic to a design traffic load for the design period; also expressed as Equivalent Axle Loads or EALs.

Erosion: The process of wearing of soil by water, wind, ice, and gravity and the detachment and movement of soil particles by same.

Evapotranspiration: The return of moisture to the atmosphere by the evaporation of water from soil and transpiration from vegetation.

Fines: Silt and clay particles in a soil, generally those passing the 0.075 mm or No. 200 sieve.

First Flush: The initial portion of a rainstorm that can flush high concentrations of accumulated pollutants into a storm drainage system. High concentrations are usually caused by antecedent dry weather conditions, which create an accumulation of pollutants on pavements washed off by the rainstorm.

Flexible Pavement: A pavement structure that maintains intimate contact with and distributes loads to its subgrade by a combination of horizontal and vertical forces. The base course materials rely on aggregate interlock, particle friction, and cohesion for stability.

Flowable Fill: Cemented aggregate produced in a concrete plant (typical maximum compressive strength of 0.7 MPa (100 psi). May also be referred to as flowable concrete fill.

Frost Action: Freezing and thawing of moisture in pavement materials and the resultant effects on their stability.

General Contractor: Party responsible to the owner for a construction contract.

Geogrids: Plastic reinforcement for pavement, including both biaxial and triaxial types, that distribute the load above by the use of tensile strength primarily to minimize deflection.

Geomembranes: Typically, EPDM (ethylene propylene diene monomer) rubber, PVC (polyvinyl chloride), or HDPE (high-density polyethylene) liners, which can prevent water from passing through them. Minimum thicknesses and methods for construction joints are critical to their successful installation with PICP systems.

Geotextiles: Woven or nonwoven fabrics made from plastic fibers used for separation, reinforcement, or drainage between pavement layers and the adjacent soils.

Gradation: Soil, sand, or aggregate base distributed by mass in specified particle-size ranges. Gradation is typically expressed in percent of mass of sample passing a range of sieve sizes. *See* ASTM C136 (2014b) for more details.

Grade: (noun) The slope of a finished surface of an excavated area, base, or pavement usually expressed in percent; (verb) to finish a surface of same manually or with mechanized equipment.

Gravel: Rounded or semirounded particles of rock that pass through a 75 mm (3 in.) sieve and are retained on a 4.75 mm (No. 4) sieve, which naturally occurs in streambeds or riverbanks that have been smoothed by the action of water. A type of soil as defined by the Unified Soil Classification System that has particle sizes ranging from the 4.75 mm (No. 4) sieve size and larger.

H-20 and HS-20: Heavy vehicle loading designation used in the design of bridges and other suspended structures. This designation for vehicle loading should not be used for pavement design.

Herringbone Pattern: A pattern where joints are no longer than the length of 1 1/2 pavers. Herringbone patterns can be set at 45° or 90°, depending on the orientation of the joints with respect to the direction of the traffic.

Hotspot: A land use that generates highly contaminated runoff with concentrations higher than those typical to stormwater runoff, usually because of a current or former land use.

Hydraulic Conductivity: The property of soils and rocks that describes the east with which a fluid (usually water) can move through the spaces or fractures.

Hydrologic Soil Group: The soils classification system developed by the U.S. Soil Conservation Service (now the Natural Resources Conservation Service) that categorizes soils into four groups, A through D, based on runoff potential. A soils have high permeability and low runoff, whereas D soils have low permeability and high runoff.

Impervious Cover: Any surface in the built environment that prohibits percolation and infiltration of rainwater into the ground below; a term that commonly refers to pavements and roofs.

Infiltration: The downward movement of water through an open-graded, crushed stone base into the soil beneath or through the soils.

Infiltration Rate: The rate at which stormwater moves from the surface into the soil, typically measured in inches per hour, or millimeters, centimeters, or meters per second. It is critical to note that infiltration rate and hydraulic conductivity are two different concepts and that conversion from one parameter to another cannot be done through unit conversion (*see* Hydraulic Conductivity and Saturated Hydraulic Conductivity). Typically, saturated hydraulic conductivity is converted to an infiltration rate to be used as a design input based on approximate relationships, which may include an infiltration reduction factor (or safety factor).

Infiltration Reduction Factor: A safety factor applied to the infiltration capacity of the permeable pavement subgrade to account for future deposition of fine particles on top of the subgrade which can reduce the ability or rate of infiltration into the subgrade.

Installer: Party responsible for the installation of the permeable pavement system. May be a subcontractor to the general contractor.

Interlock: Frictional forces between paving units that prevent them from rotating, or moving horizontally or vertically in relation to each other.

Interlocking Concrete Pavement: A system of paving consisting of segmental concrete paving units with either rectangular or indented shapes manufactured from concrete. Either type of shape is placed in an interlocking pattern, compacted into coarse bedding material, the joints filled with joint material, and compacted again to start interlock. The paving units and bedding material are placed over an unbound or bound aggregate layer. Also called concrete block pavement.

Joint: The space between concrete paving units typically filled with sand for conventional interlocking concrete pavement or small, open-graded aggregate for permeable interlocking concrete pavement.

Jointing Aggregate: Small aggregates swept into the openings between the pavers. The aggregate size varies based on the joint width. Typical sizes are ASTM No. 8, 89, or 9 stone.

Karst Geology: Regions of the Earth underlain by carbonate rock typically with sinkholes and/or limestone caverns. Infiltration practices are strongly discouraged in these areas since they can cause sinkholes.

Laying Face: The open edge or face of a concrete paver laying pattern under construction not yet constrained by a curb, building, or other restraint.

Laying Pattern: The sequence of placing pavers where the installed units create a repetitive geometry. Laying patterns may be selected for their visual or structural benefits.

Life Cycle Analysis: Assessing the environmental impacts from all the stages of a product's life, including raw material extraction, materials processing, manufacture, distribution, construction, use, repair and maintenance, and disposal or recycling.

Life Cycle Cost Analysis: A method of calculating all costs anticipated over the life of the pavement, including construction costs. Discounted cash-flow methods are generally used, typically with calculation of present worth and annualized cost. Factors that influence the results include the initial costs, assumptions about maintenance and periodic rehabilitation, pavement user and delay costs, salvage value, inflation, discount rate, and the analysis period. A sensitivity analysis is often performed to determine which variables have the most influence on costs.

Manufacturer: Party responsible for the fabrication and delivery of unit concrete pavers to the site.

Mechanical Installation: The use of machines to lift and place layers consisting of many pavers in their final laying pattern. The machines increase the rate of paving, as compared to manual placement of unit pavers (i.e., one at a time).

Mechanistic-Empirical Design: Analysis of the structural responses of applied loads through modeling stresses and strains in a pavement structure, and validation with full-scale load testing.

Modified Proctor Test: A variation of the standard Proctor test used in compaction testing, which measures the density–moisture relationship under a higher compaction effort.

Municipal Separate Storm Sewer Systems, or MS4s: A conveyance or system of conveyances owned by a state, city, town, village, or other public entity that discharges to waters of the United States and is designed or used to collect or convey stormwater. These systems include storm drains, pipes, ditches, streams, and lakes. MS4s do not include combined sanitary and storm sewers, nor do they include publicly owned treatment works (sewage treatment plants).

Observation Well: A perforated pipe inserted vertically into an open-graded base or subbase used to monitor the water level and determine its infiltration rate.

One-Hundred-Year Storm: A 100-year recurrence interval (*see* Recurrence Interval and Return Period). A rainfall event that, on average, has a 1% probability of occurring in any given year.

One-Year Storm: A one-year recurrence interval (*see* Recurrence Interval and Return Period). A rainfall event that, on average, has a 100% probability of occurring in any given year.

Open-Graded Aggregate: A crushed stone aggregate material with limited fine particles. The void spaces between the aggregates can store water and allow free drainage.

Outlet: The point at which water is discharged into a stream, lake, river, storm drain, or conveyance channel or location.

Pavement Structure: A combination of subbase, base course, and surface course placed on a subgrade and designed to support traffic loads.

Peak Discharge Rate: The maximum instantaneous flow rate from a detention or retention pond, open-graded base, pavement surface, storm drain, stream, or river usually related to a specific storm event.

Performance Period: The period of time an initial pavement structure will last before requiring rehabilitation. The performance period is the time from initial serviceability as a new, reconstructed, or rehabilitated pavement structure to its terminal serviceability requiring rehabilitation.

Permeability: The rate a fluid (in this case, water) passes through a porous medium, usually expressed in calculations per specific ASTM or AASHTO tests, and typically expressed in inches per hour or meters per second.

Permeable Interlocking Concrete Pavement (PICP): A paving system consisting of segmental concrete paving units with rectangular or indented shapes manufactured from concrete. Either type of shape is placed in an interlocking pattern, compacted into a highly permeable bedding layer consisting of small aggregate, the joints filled with a highly permeable aggregate compacted again to start interlock. The paving units and bedding material are placed over a highly permeable open-graded base typically 200 mm (4 in.) thick of aggregates ranging in size from 20 mm down to 3 mm. This layer is placed over an open-graded subbase consisting of larger sized aggregates, an unbound or occasionally a bound layer. Sand is not used within the PICP pavement structure.

Permeable Pavement: A surface with openings capable of passing water through it while supporting pedestrian and vehicles (e.g., permeable interlocking concrete pavement).

Pervious Pavement: A surface capable of accepting water and supporting pedestrian and vehicles (e.g., pervious concrete).

Pervious or Permeable Surfaces and Cover: Surfaces that allow the infiltration of rainfall, such as vegetated areas.

Porosity: Volume of voids in a base divided by the total volume of the base.

Porous Pavement: A surface full of pores supporting pedestrians and vehicles (e.g., porous asphalt).

Present Serviceability Index (PSI): Developed for the AASHO Road Test and represents the ride quality rating that required a panel of observers to ride in an automobile and provide an assessment of smoothness on a scale of 0 to 5.

Pretreatment: Best management practices (BMPs) that provide storage and filtering of pollutants before they enter another BMP for additional filtering, settling, and/or processing of stormwater pollutants.

R-Value: A value given to a saturated, compacted test specimen of soil when it exudes from the testing device under a 2-MPa (190-psi) load. *See* ASTM D2844 or AASHTO T-190 for more details.

Recurrence Interval: The recurrence interval is based on the probability that the given event will be equaled or exceeded in any given year **(***see* Return Period). Rainfall recurrence intervals are based on the magnitude and the duration of a rainfall event.

Reservoir Layer: The portion of a permeable pavement designed hydrologically to detain or retain water, typically limited to the subbase and base layers.

Resilient Modulus: Testing that characterizes load (pressure or stress) to deformation (strain) and suggests soil (or base) resistance to rutting. Modulus is express in megapascals or pounds per square inch.

Retention Pond: A body of water that collects runoff and stays full permanently. Runoff flowing into the pond that exceeds its capacity is released into a storm drain, stream, lake, or river.

Return Period: An estimate of the likelihood of a storm event occurring, usually expressed in years. (*See* One-Hundred-Year Storm, One-Year Storm, and Recurrence Interval.)

Run-On: The portion of rainfall that drains onto a site.

Runoff: The portion of rainfall that drains away from a site.

Runoff Coefficient: Ratio of surface runoff to rainfall expressed as a number from 0 to 1.

Sailor Course: A paver course where lengths of rectangular pavers are laid parallel (lengthwise) to the edge restraint.

Sand: A soil larger than the 0.075 mm (No. 200) sieve and passing through the 4.75 mm (No. 4) sieve, according to the Unified Soil Classification System.

Saturated Hydraulic Conductivity: A quantitative measure of a saturated soil's ability to transmit water when subjected to a hydraulic gradient.

Sediment: Soils transported and deposited by water, wind, ice, or gravity.

Serviceability: The ability of the pavement to serve the type of traffic (e.g., pedestrians, cars, trucks, buses, and other heavy vehicles) that uses the facility. The primary measure of serviceability is the present serviceability index (PSI), which ranges from 0 (very poor road) to 5 (perfect road).

Sheet Flow: The laminar movement of runoff across the surface of the landscape.

Silt: A soil with no more than 50% passing through the 0.075 mm (No. 200) sieve that has a low plasticity index in relation to the liquid limit, according to the Unified Soil Classification System.

Soldier Course: A paver course where lengths of rectangular pavers are laid perpendicular to the edge restraint.

Spacer Bars: The small protrusions on the sides of pavers keeping them uniformly spaced apart while compacting the pavement surface. They facilitate filling joints with aggregate and help minimize chipping and spalling during installation. Mechanically installed pavers should have spacer bars.

Storm Water Control Measure (SCM): Elements and best management practices that are designed and implemented to eliminate or reduce contact or exposure of pollutants to stormwater or remove pollutants from stormwater prior to discharge from the facility.

Structural Number (SN): A numerical value used by AASHTO to assess the structural capacity of a pavement to handle ESALs and soil subgrade strength.

Subbase: The layer or layers of specified or selected material of designed thickness placed on a subgrade to support a base course. Aggregate subbases are typically made of stone pieces larger than those in bases.

Subgrade: The soil upon which the pavement structure is constructed.

Swale: A small, linear topographic depression that conveys runoff.

Time of Concentration: The time runoff takes to flow from a drainage area's most distant point to the point of discharge.

Total Maximum Daily Load (TMDL): A term in the U.S. Clean Water Act describing the maximum amount of a pollutant that a body of water can receive daily without significantly impairing the water quality or health of the existing aquatic ecosystem.

Treated Base: An aggregate base or subbase combined with cement, asphalt, or other material added to increase its stiffness and structural capacity.

Unified Soil Classification System: Method of describing the type and physical properties of soils. *See* ASTM D2488-17.

Wearing Course: The top concrete paver surface of a permeable pavement system, which is subjected to traffic loads directly.

2.3 Consensus Standards and Other Referenced Documents

This section lists the consensus standards and other documents that shall be considered part of this standard to the extent referenced in the chapters. Those referenced documents identified by an asterisk (*) are not consensus standards; rather, they are documents develop within the industry and represent acceptable procedures to the extent referred to in the specified section.

***AASHTO GDPS-4**, *Guide for Design of Pavement Structures*, 4th Ed., American Association for State Highway and Transportation Officials, 1993.

AASHTO T307, *Standard Method for Testing Resilient Modulus*, American Association for State Highway and Transportation Officials, 1999.

AASHTO T-11, *Standard Test Method for Materials Finer than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing*. American Association for State Highway and Transportation Officials, 2005.

***AASHTO M-43**, *Sizes of Aggregate for Road and Bridge Construction*, American Association for State Highway and Transportation Officials, 2013a.

***AASHTO T-193**, *Standard Method of Test for The California Bearing Ratio*, American Association for State Highway and Transportation Officials, 2013b.

*AASHTO T 19M/T 19-14, *Standard Test Method for Bulk Density ("Unit Weight") and Voids in Aggregate*, American Association for State Highway and Transportation Officials, 2014a.

***AASHTO T-27**, *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates*, American Association for State Highway and Transportation Officials, 2014b.

***AASHTO M-288-15**, *Standard Specification for Geotextile for Highway Applications*, American Association for State Highway and Transportation Officials, 2015a.

***AASHTO T-96**, *Standard Test Method for Resistance to Degradation of Small-size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine*, American Association for State Highway and Transportation Officials, 2015b.

AASHTO HMAGGR-17, *Standard Specifications for Transportation Materials and Methods of Sampling and Testing*. American Association for State Highway and Transportation Officials, 2017a.

AASHTO T-180, *Standard Method of Test for Moisture–density Relations of Soils Using a 4.54 kg (10 lb) Rammer and a 457 mm (18 in.) Drop*. American Association for State Highway and Transportation Officials, 2017b.

***ABA**, *Architectural Barriers Act – United States Access Board*, 1968.

***ADA**, *Americans with Disabilities Act*, 1990.

***ADAAG**, *ADA Accessibility Guidelines*, 1990.

***AODA**, *Accessibility for Ontarians with Disabilities Act*, 2005.

***ASTM D3385**, *Standard Test Method for Infiltration Rate of Soils in Field using Double-ring Infiltrometer*. Annual Book of ASTM Standards, Vol. 04.08. ASTM International, 2009.

***ASTM D2487**, *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)*. Annual Book of ASTM Standards, Vol. 04.08. ASTM International, 2011a.

ASTM E2835, *Standard Test Method for Measuring Deflections using a Portable Impulse Plate Load Test Device*. Annual Book of ASTM Standards, Vol. 04.03. ASTM International, 2011b.

***ASTM D448**, *Standard Classification for Sizes of Aggregate for Road and Bridge Construction*, Annual Book of ASTM Standards, Vol. 04.03. ASTM International, 2012a.

***ASTM D698**, *Standard Test Methods for Laboratory Compaction Characteristics of Soil using Standard Effort [12,400 ft-lbf/ft^3 (600 kN-m/m^3)]*. Annual Book of ASTM Standards, Vol. 04.08. ASTM International, 2012b.

***ASTM D2844/D2844M-13**, *Standard Test Method for Resistance R-value and Expansion Pressure of Compacted Soils*. Annual Book of ASTM Standards, Vol. 04.08. ASTM International, 2013b.

*ASTM C131/C131M-14**, *Standard Test Method for Resistance to Degradation of Small-size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine*. Annual Book of ASTM Standards, Vol. 04.02. ASTM International, 2014a.

*ASTM C136/C136M-14**, *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates*. Annual Book of ASTM Standards, Vol. 04.02. ASTM International, 2014b.

*ASTM C1781**, *Standard Test Method for Surface Infiltration Rate of Permeable Unit Pavement System*. Annual Book of ASTM Standards, Vol. 04.05. ASTM International, 2015.

***ASTM C33/C33M-16**, *Standard Specification for Concrete Aggregates*. Annual Book of ASTM Standards, Vol. 04.02. ASTM International, 2016a.

ASTM C535, *Standard Test Method for Resistance to Degradation of Large-size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine*. Annual Book of ASTM Standards, Vol. 04.02. ASTM International, 2016b.

*ASTM C936/C936M-16**, *Standard Specification for Solid Concrete Interlocking Paving Units*. Annual Book of ASTM Standards, Vol. 04.03. ASTM International, 2016c.

ASTM C979/C979M-16, *Standard Specification for Pigments for Integrally Colored Concrete*. Annual Book of ASTM Standards, Vol. 04.05. ASTM International, 2016d.

ASTM C1645, *Standard Test Method for Freeze–Thaw and De-icing Salt Durability of Solid Concrete Interlocking Pavement Units*, ASTM International, 2016e.

***ASTM D1883**, *Standard Test Method for CBR (California Bearing Ratio) of Laboratory Compacted Soils.* Annual Book of ASTM Standards, Vol. 04.08. ASTM International, 2016f.

ASTM D5126-16e1, *Standard Guide for Comparison of Field Methods for Determining Hydraulic Conductivity in Vadose Zone*. Annual Book of ASTM Standards, Vol. 04.08. ASTM International, 2016g.

***ASTM C29/C29M**, *Standard Test Method for Bulk Density (Unit Weight) and Voids in Aggregate*. Annual Book of ASTM Standards, Vol. 04.02. ASTM International, 2017a.

*ASTM C117-17**, *Standard Test Method for Materials Finer than 75-µm (No. 200) Sieve in Mineral Aggregates by Washing*, ASTM International, 2017b.

ASTM C140, *Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units*. Annual Book of ASTM Standards, Vol. 04.05, ASTM International, 2017c.

ASTM C1701/1701M, *Standard Test Method for Infiltration Rate of In Place Pervious Concrete*. Annual Book of ASTM Standards, Vol. 04.02, ASTM International, 2017d.

ASTM D6938, *Standard Test Method for In-place Density and Water Content of Soil and Soil-aggregate by Nuclear Methods (Shallow Depth)*. Annual Book of ASTM Standards, Vol. 04.09. ASTM International, 2017e.

CSA A23.1/A23.2, *Concrete Materials and Methods of Concrete Construction/Test Methods and Standard Practices for Concrete*, Canadian Standards Association, 2014a.

CSA A231.2, *Precast Concrete Pavers*, Canadian Standards Association, 2014b.

CSA A23.2A (A23.2-14), *Sieve Analysis of Fine and Coarse Aggregates Concrete Materials and Methods of Concrete Construction/Test Methods and Standard Practices for Concrete*, Canadian Standards Association, 2014c.

CSA A23.2-10A, *Density of Aggregate. Concrete Materials and Methods of Concrete Construction*, Canadian Standards Association, 2014d.

CSA A23.2-16A, *Resistance to Degradation of Small-size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine (for Aggregate ≤ 40 mm). Concrete Materials and Methods of Concrete Construction/Test Methods and Standard Practices for Concrete*, Canadian Standards Association, 2014e.

CSA A23.2-17A. *Resistance to Degradation of Large-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine (for Aggregate > 40 mm). Concrete Materials and Methods of Concrete Construction/Test Methods and Standard Practices for Concrete*, Canadian Standards Association, 2014f.

Chapter 3

PRELIMINARY INFORMATION FOR THE DESIGN OF PERMEABLE PAVEMENTS

Permeable interlocking concrete pavement (PICP) can provide a durable and cost-efficient pavement. PICP is also recognized as an effective best management practice or stormwater control measure (SCM). As for any pavement and SCM, proper design, construction, and maintenance procedures are required. The engineer is responsible for applying sound scientific and engineering methods to PICP design. The results from application of those methods are only as good as the data used. Chapter 4 describes data requirements and the structural and hydrologic design elements required.

3.1 PROJECT SUITABILITY

There are several key factors that should be considered when determining the suitability of a project for the use of permeable pavement systems. Based on their importance in overall decision making, these factors may affect the decision to use permeable pavements for a particular project. A summary of some suitability factors for consideration appear in Table 3-1. Key factors may not have equal weight on making a decision to use PICP.

3.2 PRIMARY PICP DESIGN OPTIONS

There are three primary PICP hydraulic design options: full infiltration, partial infiltration, and no infiltration. Depending on the type of the subgrade soils, the PICP system design can include capture, detention or retention, filtration, and infiltration of stormwater into the subgrade.

Full-infiltration designs are used where subgrade soils have high to moderate infiltration rates (Fig. 3-1a). Although water storage is available within the open-graded aggregates, all of the stormwater entering the PICP infiltrates into the subgrade. Rainfall depths exceeding the storage and infiltration capacity drain out of the PICP via overflow pipes.

Partial-infiltration designs are used where subgrade soils have low infiltration rates and when all of the water cannot drain into the subgrade within a reasonable period of time, especially during extreme storm events. A perforated outlet pipe (also called an underdrain) should be included through which excess water discharges to a surface water feature or underground stormwater collection system.

No-infiltration designs prevent infiltration into the subgrade. This goal is typically achieved by using a geomembrane to encapsulate the bottom and sides of the pavement system. Water is detained within the open-graded aggregates until such time as it is discharged through the underdrain. Outlet pipes can be raised with a sump underneath to enhance retention time and runoff reduction benefits.

Although the subgrade soil infiltration rate may be the predominant factor in selecting a design option, the base and/or subbase thickness is based on structural as well as hydrologic requirements (see Section 3.3). There are other design elements that may affect the selection of a design option. Examples of some design elements to consider are provided in Table 3-2.

3.3 PICP SYSTEM COMPONENTS

PICP differs from conventional pavements. For conventional flexible pavements, all layers are constructed as densely as possible to ensure maximum load-carrying capacity. Layer thicknesses are determined from the subgrade strength and anticipated traffic. PICP must be designed for hydrologic capacity as well as structural capacity since infiltration through the layers is required. Typical PICP components are shown in Fig. 3-2.

Edge Restraint: All PICP shall be designed with edge restraints (typically concrete curbs) around the perimeter. Buildings and other structures may serve as restraints. Edge restraints help maintain rotational and horizontal interlock of the paving units on the surface resulting from dynamic vehicular wheel loads such as turning, braking, and accelerating. Pedestrian pavements may use other types of edge restraint systems.

Concrete Pavers: Solid paving units with molded joints or openings that create an open area (joint area) between unit pavers typically ranging from 5 to 15% of the total surface area. Concrete pavers should conform to the latest industry standards (ASTM/CSA 2016c). Surface openings are filled with permeable joint material to allow water to freely infiltrate into and through the pavement surface. For vehicular traffic, pavers must have an aspect ratio (length divided by thickness) less than or equal to 3 and a minimum thickness of 80 mm (3 1/8 in.). Pedestrian-only areas may use 60 mm (2 3/8 in.) thick pavers.

Permeable Joint Material: Permeable, open-graded crushed stone, typically ASTM No. 8, 89, or 9. Selection of aggregate sizes depends on the joint widths created by the assembled concrete pavers [(e.g., larger gradation materials (ASTM No. 8)] are typically used for wider joint openings. Consideration should also be given to the requirements of the Architectural Barriers Act (ABA), Americans with Disabilities Act (ADA), ADA Accessibility Guidelines (ADAAG), and Accessibility for Ontarians with Disabilities Act (AODA), or other accessibility guidelines as required when considering the concrete paver and joint material types.

Open-Graded Bedding Course: This permeable layer of crushed stone bedding is typically 50 mm (2 in.) thick before compaction and provides a level bed for the concrete pavers. It consists of small, open-graded aggregate, typically ASTM No. 8, 89, or 9 stone or similar sized material. This layer is not typically considered part of the reservoir for water storage within the pavement structure.

Open-Graded Base Reservoir: This is an aggregate layer, sometimes referred to as a "choker layer," typically 100 mm (4 in.) thick for vehicular traffic and made of crushed stone primarily graded from 25 mm down to 13 mm (1 in. down to 1/2 in.). For relatively thin pavements, this layer may constitute the only open-graded layer. Besides storing water, this high-infiltration-rate layer provides a gradational transition between the bedding and subbase layers. The stone size is typically ASTM No. 57 or similar sized material.

Open-Graded Subbase Reservoir: The stone sizes of this layer are larger than those for the base, primarily graded 75 mm down to 50 mm (3 in. down to 2 in.), typically ASTM No. 2, 3, or 4 stone. As with the base layer, water is stored in the voids between the stones. The subbase layer thickness depends on water storage requirements and traffic loads. A subbase layer may not be required in pedestrian or residential driveway applications. In such instances, the base layer thickness may be increased to provide water storage and support. Other base and subbase materials are discussed in subsequent sections.

Filter Layer: If enhanced water quality benefits are desired from the permeable pavement installation, a filter layer may be specified within the pavement structure. This filter layer may be placed within the pavement structure above the subgrade, or a portion of the base or subbase layer may be "filled" with finer filter layer aggregate. The filter filled portion of the base or subbase layer has reduced porosity, which must be considered in the hydrologic and structural design of the pavement.

Overflow: Subsurface piping or underdrains may be designed as an overflow to accommodate some of the subsurface water flow during larger storm events. The outflow may be directed toward an outlet structure, storm drains, drainage ditches, swales, or stormwater ponds during peak water flow periods.

Utility Structures: Utility access covers, including cleanouts, should be surrounded with concrete collars. They should be square or rectangular and should fit into the surrounding paver laying pattern. The depth of the "collar" should be on the order of 200 to 250 mm (8 to 10 in.) with at least 100 mm (4 in.) of concrete around the utility structure that it surrounds. The thickness and reinforcement should be as required to meet or exceed the PICP pavement strength with an appropriate factor of safety. Additionally, utility structures should be avoided within 1 m (3 ft) of the nearest edge restraints for PICP pavement.

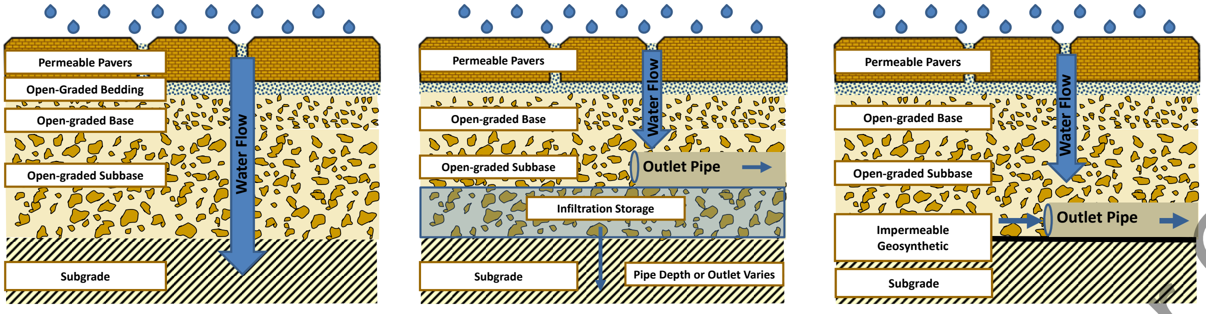
Geosynthetic: A geotextile, geogrid, or geomembrane incorporated into a site-specific design as a design element required by the engineer.

- Geotextile: A fabric (woven or nonwoven) that separates the subbase from the subgrade to prevent migration of soil. Also, typically used as a vertical separator between the permeable pavement and adjacent nonpermeable pavement or soils to prevent the migration of fines into the permeable pavement.
- Geogrid: A flexible grid-type structure used to improve the structural capacity of a pavement by supporting the pavement layers above and distributing the resulting load over a wider base. A geogrid typically works in tension, thereby better providing support to pavement layers above it when placed over somewhat flexible or yielding subbase or subgrade.
- Geomembrane: A synthetic barrier that resists the passage of liquids or gases. Often referred to as an "impermeable liner," it is used when infiltration of stormwater from the PICP into the subgrade is not desired. Common materials include polyvinyl chloride (PVC), high-density polyethylene (HDPE), and other types of rubber or plastic liners that can be made watertight.

Subgrade: The soil immediately beneath the aggregate base or subbase. This soil may be imported compacted material or native soil appropriate for the design strategy. This layer may be compacted for additional structural support and to reduce the risk of rutting over time. Assessing density and infiltration of compacted subgrades is covered elsewhere. If not compacted, the reduced density from no or little compaction and the resulting loss of support must be considered in the structural design of the pavement.

Underdrain: Underdrains facilitate water removal from the base and subbase as required. The underdrains are typically perforated or slotted pipes resistant to crushing damage that "daylight" to a surface water feature (swale or stream) or connect to a larger underground stormwater collection system. Another design option to which underdrains connect is plastic or concrete stormwater storage chambers. These chambers are often designed within the aggregate subbase layer and can store increased amounts of runoff.

Discharge Outlet Design: Outflow details depend on the design goals for the PICP. Various designs for the discharge of water do not infiltrate into the subgrade. These designs may include raised outlet pipes or flow restrictors to infiltrate or detain stormwater in the system. The design of discharge is similar to outlet structures used for conventional stormwater management facilities, including weirs and orifices.



(a) (b) (c)

Figure 3-1. Infiltration Examples: (a) Full Infiltration; (b) Partial Infiltration; and (c) No Infiltration

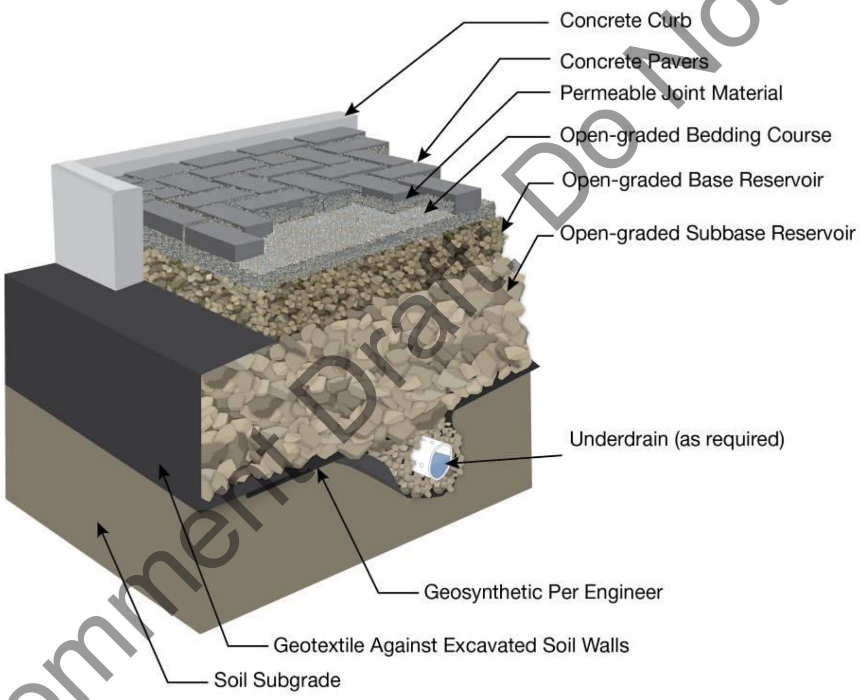


Figure 3-2. Typical Components of PICP

Source: ICPI 2017c; reproduced with permission from Interlocking Concrete Pavement Institute.

Table 3-1. Factors in Determining the Suitability for Use of Permeable Pavement Systems

Factors to Consider	*Description*
Cost efficiency and financing	PICP can be cost-effective when the total cost of pavement, drainage infrastructure, stormwater quality management, potential groundwater recharge, and land costs are considered.
Life cycle cost	The total life cycle cost of the PICP system can be lower than the cost of conventional pavement systems.
Environmental approval process	Desirability for stormwater quality and quantity management can drive regulatory acceptance of, and even advocacy for, PICP. Limits on impervious cover may also determine the use of permeable pavements, which is generally considered a pervious or permeable surface by local regulations. Barriers to PICP use may include lack of confidence in performance locally for stormwater volume control or pollutant reduction.
Safety	PICP may be able to provide safety features such as traffic calming, colored units for permanent marking of parking spaces, driving lanes, and direction-finding; reduced flooding; reduced ice formation; and reduced slip hazards from freezing temperatures.
Site grades	A pavement with steeply sloped surfaces and subgrade (e.g., >5%) may be less effective at promoting infiltration and water storage than a level pavement system (e.g., <2%). Significant longitudinal grades may require additional design features, such as intermittently spaced berms, check dams, walls, or baffles in the subbase that create a stepped or terraced system with level sections. These features can promote infiltration to achieve design goals and prevent water from exiting the pavement surface at the low elevation end of the system.
Depth of water table	Permeable pavements designed for full or partial infiltration may not function properly in areas where the seasonal high water table is close to the bottom of the pavement system. The minimum separation from the high water table for full and partial-infiltration systems is typically 0.6 m (2 ft) to achieve infiltration benefits. This separation allows infiltration of collected water into the subgrade. Note that this criterion is not intended to address potential for groundwater contamination as a result of high groundwater or other soil factors (see Groundwater contamination). High groundwater does not exclude the use of permeable pavement, but it does affect infiltration capabilities.
Geotechnical aspects	Site-specific geologic and soil conditions, such as presence of organics, fill soils, clay soils, karst geology, bedrock, and subgrade prone to infiltration, may pose a range of geotechnical risks, such as reduced subgrade support, potential for subgrade frost heave, settlement, shrinking or swelling, localized scouring, sinkholes, or lack of proper infiltration. Geotechnical risks may introduce added design complexity and may necessitate the use of an underdrain, subgrade stabilization, and/or a geomembrane in the pavement section.
Groundwater contamination	A variety of factors influence the potential for stormwater sources to contaminate groundwater, including soil characteristics, depth to groundwater, traffic volume, existing soil contamination sources, site use (e.g., vehicle salvage yards, fueling stations, or maintenance facilities), and salt application or other chemicals for deicing.
Stringent receiving water quality standards	The presence of and need to protect nearby aquatic resources may provide incentives for the use of permeable pavements. For some protected watersheds, cold water streams, and other receiving waters with stringent water quality standards, the level of treatment provided by permeable pavements (for water discharged from underdrains) in some cases may not provide adequate protection from stormwater quality effects. In cases where infiltration is not feasible, underdrains may need to convey water infiltrated

	through the PICP to a surface discharge location or storm drain, and additional treatment may be needed to protect receiving waters.
Winter maintenance	Procedures for snow and ice removal are similar to those for conventional pavements. The need for deicing chemicals may be reduced when appropriate snow and ice removal procedures are followed. PICP facilitates continued surface drainage during freeze and thaw cycles, as compared to traditional, impermeable pavement systems.
Sand use for winter maintenance	If not removed in the spring, the accumulation of fine sand during the winter may clog permeable pavement surfaces, resulting in reduced surface infiltration. The use of coarser sand or jointing aggregate (ASTM No. 8 or 9) for winter traction can reduce the potential for clogging.
Construction sediment control	Permeable pavement systems built early in the construction sequencing may clog with sediment from construction site runoff, run-on, or vehicles. The surface may require remedial cleaning before project acceptance. A detailed erosion and sediment control plan diverting sediment sources away from the permeable pavement is essential for the long-term performance of PICP.
Low soil infiltration rates	Soil infiltration rates influence performance of PICP for volume and peak flow reduction. Systems with low native soil infiltration rates below them may need to be supplemented with an underdrain to achieve drainage goals.
Stormwater hydrologic design for quantity control	A site-specific hydrologic analysis based on site rainfall patterns, tributary drainage areas, impervious pavement geometry, flow paths, and stormwater management goals (i.e., long-term runoff volume or rate and peak flow reduction) is recommended to evaluate whether PICP meets design goals.
Stormwater hydrologic design for quality control	Permeable pavements may contribute substantially to water quality improvement (e.g., temperature or reduction of total suspended solids or of other chemical contaminants). Regulations that require stormwater quality management may significantly incentivize permeable pavement use.
Complexity of site conditions	Site constraints, such as horizontal or vertical grades, structures, or foundations; or existing infrastructure, such as curbs, retaining walls, utilities, catch basins, and proximity to septic systems or to municipal or private wells, require careful consideration in design.
Risk of flooding	Permeable pavement systems may not be capable of retaining or conveying all peak flows and volumes from larger, less frequent storms. Areas subject to frequent flooding may require supplemental drainage features or overflow locations to ensure that they adequately drain from such storm events.
Pavement surface maintenance	Sediment removal is required to maintain the designed permeability of the pavement. The frequency and type of cleaning equipment depend on the sources of sediment that may reduce pavement permeability. Stabilized contributing drainage areas can lower PICP maintenance costs.
Structural design	Design of permeable pavements for moderate to heavy axle loads or high traffic counts may require additional structural design analysis and details.
Interest in innovation and sustainability	Designs including permeable pavements can provide opportunities for innovation and sustainable benefits.
Accidental spills	For spill risk areas, such as loading docks or point source discharge areas, PICP may assist in spill containment by using a geomembrane.
Owner experience	Permeable pavement systems should be designed to address owner expectations for performance, aesthetics, inspections, maintenance, benefits, and costs.

Table 3-2. PICP System Design Assessment Steps

Design Consideration	*Assessment Step*
Site Characterization	
Drainage path	Conduct an evaluation of the drainage patterns in the surrounding area to determine its possible effect on pavement performance, i.e., contributing drainage areas, run-on to the pavement, and sediment sources.
Traffic type and patterns	Conduct an assessment of the anticipated traffic type, volume, and composition. Identify areas of heavy or concentrated loading.
Winter maintenance	Prohibit or discourage the use of sand for traction, which may clog the pavement. Limit use of sand and especially deicing chemicals if water is captured and reused.
Groundwater depth	Determine the proximity of the seasonal high groundwater elevation. In areas where the groundwater is within 0.6 to 1 m (2 to 3 ft) of the bottom of the pavement, assess the effect of water mounding on full- or partial-infiltration designs, or use a no-infiltration design.
Subsurface conditions	Assess the effect of the PICP on underground utilities and local conditions, i.e., the presence of bedrock, that may require special considerations. Note that easy access to utility repair work by removing PICP units is possible, facilitating repairs, while reducing effects on the overall pavement structure related to utility work.
Surrounding land use	Identify upslope areas of high sediment and/or contaminant-generating activities. Identify run-on potential and effects on the PICP design, inspection, and maintenance requirements.
Rainwater capture and reuse	Limit exposure to potential contaminants for systems where the stormwater is being captured for reuse.
Structural Design	
Traffic	Use the type, loading, and frequency of traffic to determine the design traffic for the pavement. Areas of concentrated heavy, frequent traffic loading may require enhanced structural design detailing.
Subgrade characteristics	Carefully evaluate the effect of saturated conditions and length of saturation time on subgrade structural capacity. Determine infiltration capacity. Determine the anticipated number of days per year that the subbase is exposed to standing water. Assess the need for compaction of the subgrade to uniform density. For low-strength (California bearing ratio < 4) soils, consider the use of geosynthetics or other stabilization techniques.
Permeable paver and bedding layer	Consider paver type, thickness, joint width and jointing materials, laying pattern, border course(s), and manual versus mechanical installation techniques as they relate to surface stability, constructability, and cost.
Open-graded base and/or subbase	Select durable and angular materials sized to interlock or choke into the underlying layer of material and maximize structural capacity and porosity for water storage. Select washed materials with very low amounts of fine material (<2% passing through 0.075 mm (No. 200) sieve).
Geosynthetics (e.g., geotextiles, geogrids, and geomembranes)	Assess the need and/or benefit of geosynthetics for separation, containment, reinforcement, and membrane protection.
Hydrologic Design	
Hydrologic analysis	Determine the appropriate design inputs for the PICP hydrologic analysis. Check local requirements.
Surface infiltration	Consider initial and long-term infiltration capacity to account for pavement clogging.
Pavement surface slope	Determine the effect of surface slope of the pavement on the ability of the pavement to infiltrate water.
Subgrade slope	Minimize subgrade slopes for infiltration designs. Consider berms, check dams, walls, and weir structures in the pavement for steeply sloped sites. Use flatter subgrade slopes where possible.
Subgrade infiltration	Determine the potential for subgrade infiltration based on soil type, density,

	and permeability. Subgrade compaction and infiltration are competing design factors; higher subgrade compaction helps support pavement traffic, and lower (or no) compaction helps maximum infiltration. Site-specific designs are recommended.
Hydraulic head	Determine if sufficient hydraulic head exists to accommodate drainage from the pavement system.
Contributing area run-on	Determine run-on water volume from contributing areas. Consider potential sediment and contaminant loads.
Run-on from roof downspouts	Determine design details necessary to accommodate water from roof downspouts and other directly connected impervious areas. Consider run-on from splash pads and scouring potential for discharge directly into the pavement base and/or subbase.
Storage capacity	Consider surface and subsurface slope and base or subbase porosity to determine effective storage capacity and the movement of water through the pavement system.
Surface overflows	Design adequate conveyance (runoff) or storage for larger storm events that may temporarily exceed surface infiltration capacity. The PICP surface (including the setting bed) should not be included as a part of the storage reservoir in the hydrologic calculations.
Subsurface overflows	In areas of frequent, high-intensity storms, drainage pipes may be placed in the upper base to prevent water from overflowing the pavement structure and saturating the PICP surface (including the setting bed).
Hydrologic losses	Consider other stormwater volume reduction processes that occur in the PICP system, such as aggregate and surface material adsorption. These processes are generally discounted in the water balance equation for PICP, but they can be included when considered significant to the design.
Underdrain and outlet details	Determine the type, number, size, location, and need for underdrains to achieve the hydrologic design criteria, retention goals, and any storage (drain down) time limit. Specify outlet details as well as cleanouts and observation wells. Underdrains may also be designed to restrict water from exiting at the pavement surface during nonwinter conditions or to meet stormwater capture and release goals, such as a maximum allowable discharge rate.
Geosynthetics (e.g., geotextiles, geogrids, or geomembranes)	Assess the need and/or benefit of geosynthetics for filtration and/or containment of stormwater. When selecting a geotextile, consider potential clogging caused by sediment migration through the PICP.

Chapter 4

PERMEABLE PAVEMENT DESIGN

4.1 GENERAL

The engineer determines pavement cross sections and base and/or subbase thicknesses by applying the design process illustrated in Fig. 4-1. The design steps on this figure are outlined below as a twofold design process.

First, the designer conducts a structural analysis as indicated on the left side of the flowchart shown in Fig. 4-1. Pedestrian and vehicular pavement design requires determining the subgrade soil characteristics and then surface and base or subbase thicknesses from established surface, base, and subbase properties. In addition, vehicular pavement design requires determining traffic loads expressed as lifetime 80 kN (18,000 lb) equivalent single axle loads (ESALs) (AASHTO 1993) or traffic index (TI) (Caltrans 2012). Typical design traffic classifications are provided in Table 4-1. The engineer then determines the thickness of the base or subbase necessary to accommodate the specific traffic loads.

Second, the engineer conducts a hydrologic analysis (water balance), as indicated on the right side of the flowchart to develop a subbase thickness based on water storage goals. Inputs and outputs for the hydrologic analysis include the following:

- Management requirements typically defined by the state, province, and/or local municipality;
- Direct rainfall onto the permeable interlocking concrete pavement (PICP) surface;
- Any run-on from adjacent surfaces (e.g., pavements, vegetated areas, or roofs) that becomes inflow through a rainfall–runoff relationship;
- Storage depths and volumes within the reservoir layers, based on the dimensions, elevations, and porosity within the stone layers, as well as pipes or other stormwater structures;
- The subgrade design infiltration rate (water volume that percolates into the subgrade during a design event); and
- Controlled discharge through underdrains or outlet structures (as required).

The engineer tabulates the total inputs, subtracts the total outputs during the storm duration, and then determines the base and subbase thicknesses required to store the remaining water. The remaining water is infiltrated into the ground over a period of days and/or discharged through underdrains as specified by the stormwater regulatory agency, usually 48 to 72 h, including the rainfall event. This analysis involves manual- or computer-calculated water movement into and out of the system over the analysis period.

The engineer then compares the estimated subbase depth, as determined for the purposes of water storage, with the design thickness calculated for structural purposes based on anticipated traffic loads, and selects the thicker pavement subbase section for the final design. It may be possible to optimize the design thickness such that the same thickness can accommodate structural and hydrologic design. Possible optimization options are presented in Section 4.3.

4.2 STRUCTURAL ANALYSIS

The structural analysis procedure is developed from full-scale accelerated load testing of PICP by the University of California Pavement Research Center (UCPRC) in 2014. The load testing, mechanistic design procedure, and design tables are in the UCPRC report, "Development and HVS Validation of Design Tables for Permeable Interlocking Concrete Pavement: Final Report." The design approach uses rutting as a function of the shear stress to shear strength ratios at the top of the subbase and the top of the subgrade. The shear stress–strength ratio was originally developed for airfield pavements where the shear stresses from aircraft loads and tire pressures are high relative to the low strengths of the subgrade materials. On permeable road pavements, subgrade materials may be wet for extended periods of time, resulting in reduced shear strengths compared to dry subgrade conditions. This design approach was selected based on a review of research literature, past research on permeable pavements by UCPRC, the results of deflection testing on in-service PICPs, and modeling these results and the pavement stresses based on pavement reactions from the full-scale load tests. The UCPRC analysis validated subbase thickness design procedure using AASHTO (1993) *Guide for Design of Pavement Structures*. This pavement design procedure is provided in Appendix D.

The full-scale load tests on PICP used 80 mm (3.125 in.) thick concrete pavers in a 90° herringbone pattern. The PICP structures were built over very weak clay soil (California bearing ratio [CBR] = 2%) exposed to repetitive passes by slowly moving dual truck wheels loaded from 25 kN (5,625 lb) to 80 kN (18,000 lb). Tests were conducted over dry, saturated, and drained subgrade and subbase conditions to determine effects on rutting. The measured load, deflection, and rutting relationships were used as a basis for modeling the shear behavior of the PICP pavers and base, and the subbase reservoir in dry and wet saturated conditions. From an iterative computational process, design tables were developed that establish the subbase thicknesses depending on the number of days that water stands in them (i.e., the number of days the subgrade is saturated). A higher number of saturation days per year increases the subbase thickness.

The following sections provide information and guidelines on characterizing traffic loads, soil strengths, subbase and base strengths, and number of days the subbase holds water. All these factors are inputs for selecting appropriate subbase thicknesses for the permeable pavement.

4.2.1 Traffic Loads

The PICP design procedure characterizes traffic loads as equivalent single axle loads (ESALs). One ESAL is represented as the application of a single 80 kN (18,000 lb) axle load. The cumulative ESALs for pavement design are estimated using a simplified method developed by the U.S. Long-Term Pavement Performance (LTPP) program. The method developed by LTPP estimates the cumulative ESALs based on the estimated average annual daily traffic (AADT). AADT is based on the functional classification or category of the road and the percentage of commercial truck traffic. In this design method, pavement damage is assumed to be caused solely by commercial truck traffic. Automobiles require thousands of load repetitions to exert the same damage to pavement as that from one pass of a commercial truck.

The design ESALs are estimated using the following equation:

$$\mathrm{ESALs} = \mathrm{AADT} \times \mathrm{DD} \times \mathrm{LD} \times \mathrm{CV} \times \mathrm{TF} \times \mathrm{DY} \times \left[\frac{(1+\mathrm{GR})^{\mathrm{DL}}-1}{\mathrm{GR}}\right] \qquad (4\text{-}1)$$

where

ESALs = Design equivalent single axle design loads over the design life;
AADT = Average annual daily traffic;
DD = Directional distribution (typically 50/50%, expressed as decimal = 0.5);
LD = Lane distribution (percent of vehicles in design lane for multiple lanes in each direction, expressed as a decimal);
CV = Percent of commercial vehicles (expressed as a decimal);
TF = Heavy vehicle load factor (0.76 for a typical municipal flexible pavement);
DY = Number of days per year of commercial vehicle use (typically 365);
GR = Traffic growth rate per year (expressed as a decimal percentage); and
DL = Pavement service life (years).

The California Department of Transportation (Caltrans) uses the traffic index (TI) to characterize traffic loads. The conversion of ESALs to TI follows:

$$\mathrm{TI} = 9.0 \times (\mathrm{ESAL}/10^{6})^{0.119} \qquad (4\text{-}2)$$

For this standard, ESALs and TI are provided for 10 levels of traffic up to a maximum of 1 million ESALs or TI of 9. The designer selects the anticipated number of ESALs or TI over the pavement design life, typically 20 years. To put 1 million ESALs into the context of road use, roadway classifications and their typical design ESALs or TI levels are shown in Table 4-1. A maximum of 1 million ESALs indicates that the most intense use for PICP is minor collector streets.

Most bridge structures in the United States are designed to accommodate a standard truck load. This load is designated H-20 or HS-20. The "20" in this designation stands for a 20 ton (18 metric ton) vehicle [4 tons (3.6 metric tons)] on the steering axle and 16 tons (14.5 metric tons) on the drive axle). The "S" stands for a semitrailer combination, which adds an additional 16 tons (14.5 metric tons) for the third axle to indicate a 36 ton (32.7 metric ton) vehicle. The designation of H-20 loading is used for bridge structures to ensure that the bridge will not be overloaded and fail. The use of the H-20 designation when referring to the design of pavements is incorrect. Pavements subjected to vehicular traffic are not designed for the impact of a single vehicle on the pavement. Pavements do not typically fail "catastrophically" but rather gradually to the application of many tens of thousands or even millions of axle loads.

4.2.2 Soil Subgrade Characterization

The pavement design should be based on site-specific soil properties. Therefore, testing and evaluation of in situ soils for a specific project location are highly recommended. Saturated soil subgrade strength should be assessed for all pavement designs including those using an impermeable liner over the subgrade for hydrologic design and encasing the sides of the structure. The soil strength shall be evaluated for each project from appropriate ASTM or AASHTO test methods, or other local standards. Soil strengths are characterized using laboratory testing to determine resilient modulus (AASHTO 1999) or through surrogate test methods such as the California bearing ratio (CBR) (ASTM 2016f, AASHTO 2013b) or R-value, ASTM (2013b). The CBR test should be a 96-h soaked test to represent subgrade strength in its weakest condition. CBR and R-values are correlated to resilient modulus, M_R, using Eqs. (4-3) and (4-4):

$$M_R \text{ in psi} = 2{,}555 \times (\text{CBR})^{0.64} \qquad M_R \text{ in MPa} = 17.61 \times \text{CBR}^{0.64} \qquad (4\text{-}3)$$

$$M_R \text{ in psi} = 1{,}155 + 555 \times R \qquad M_R \text{ in MPa} = (1{,}155 + 555 \times R)/145 \qquad (4\text{-}4)$$

The characterization of the compacted subgrade is essential if a PICP design is infiltrating some water, typically using a partial-infiltration design with underdrains. The designer should establish the relationship between in situ soil infiltration rate and in situ soil density achieved during construction. Establishing this relationship is achieved by obtaining laboratory Proctor density results from soils sampled from test pits within the site. Soil samples should be taken at the anticipated, approximate bottom elevation of the aggregate subbase. The designer should then return to the test pits, compact the soil to a given density, presoak the test areas, and then conduct infiltration tests on the compacted soils to estimate the infiltration rate.

The designer is cautioned that site excavation can present a greater range of soil properties than those tested or estimated. Therefore, soil subgrade strength should be confirmed at the end of excavation based on encountered subgrade conditions. The resilient modulus should be adjusted as necessary to ensure that the pavement design has adequate structural capacity for the anticipated traffic (ESALs). As a surrogate for resilient modulus, laboratory Proctor densities can be determined at a range of 9 h soaked CBR values. A target density can be used when compacting soil on the site. The target density is related to CBR, and that value can be correlated to resilient modulus using Eq. (4-3). The resilient modulus value is required for determining the subbase thickness after the target density (and corresponding CBR) is reached via compaction in a test pit or pits on the site, then the infiltration rate can be tested. Average infiltration values can be used for the hydrologic design and can be adjusted downward to account for sedimentation over time.

This process aids the designer in understanding the relationship between subgrade infiltration and the structural capacity (soil stiffness) required to support the design traffic. For example, a resilient modulus (or CBR) determined at a soil compaction level of 95% of the standard Proctor maximum dry density has lower infiltration capacity and higher structural capacity than a resilient modulus (or CBR) determined at a soil compaction level of 90%. If the in situ density is lower than the design density, the design resilient modulus is likely to need to be decreased, which decreases the structural capacity of the pavement, especially when the soil is saturated. The results require a thicker pavement structure.

A key design input that must be estimated is the number of days per year when there is water standing in the subbase, i.e., when the subgrade is saturated. This number is used to determine the subbase thickness for the permeable pavement. Based on full-scale testing, the tables account for weakening of the subgrade caused by extended saturation periods. This design input is not considered in conventional pavement design because such pavements typically do not experience the extent of saturation typical to permeable pavements. Section 4.2.11 provides detailed procedures for using the number of days per year that water stands in the subbase to determine the subbase thickness.

4.2.3 Expansive Soils and Other Considerations

When expansive clay soils are present, a no-infiltration PICP design may be necessary. Soil expansion may also be reduced or eliminated by removal and replacement of subgrade soil materials, stabilizing with additives such as lime or cement, and/or the use of a geomembrane under the base and/or subbase to protect the subgrade from expansion. This standard assumes that expansive soils are addressed by the engineer and are reflected in the design subgrade strength value.

Construction on fill soil presents the risk of unacceptable settlement and potential for pavement and/or fill slope failure. Preloaded or ballasted, engineered fill soils should be considered in the design stage, as well as adjustments to the pavement elevation over its service life. Alternatively, aggregate materials may be used as fill materials. Designs over fill soils may require the use of a geomembrane under the base or subbase.

When designing PICP in areas with karst geology, high groundwater, wetlands, source water protection, and brownfield development areas, additional site explorations and design inputs are required from suitably qualified engineers specializing in these types of conditions.

4.2.4 Aggregate Base and/or Subbase

The majority of PICP designs use open-graded aggregates to provide structural and reservoir capacity for the pavement. Aggregates used in vehicular applications should be crushed, hard, and angular and should contain a very low fines content. Jointing, bedding, base, and subbase aggregates used in vehicular PICP applications should have a minimum 90% fractured faces and a minimum Los Angeles (LA) abrasion < 40 per ASTM (2016b) or AASHTO (2015b), of the same title, and ASTM (2012a) for aggregate larger than 37.5 mm (1 1/2 in.).

Open-graded aggregates for PICP are typically specified with ≤ 2% passing the 0.075 mm (ASTM No. 200) sieve with typical densities of 1,522 to 1,922 kg/m^3 (95 to 120 lb/ft^3). Although a maximum of 2% passing the 0.075-mm (ASTM No. 200) sieve size is recommended, aggregates with a higher percentage of fines may be permitted by the design engineer if experience dictates that it will not affect the structural or hydrological performance of the permeable pavement. Washing aggregates may be necessary to achieve this fines content. Fines not removed from aggregates tend to be washed through the PICP system and may clog the subgrade, reducing its infiltration capacity. When deposited on top of the subgrade, fines can reduce its infiltration capacity. For this reason, recycled concrete aggregates are not recommended in PICP, because they can generate excessive fines from handling, placement, and compaction. In addition, recycled concrete aggregates may not meet the abrasion loss recommendations for use in vehicular traffic.

Porosity of the aggregate when compacted (i.e., the volume of voids divided by the total volume of the base and/or subbase) should be selected to reflect water storage objectives. Porosity can be approximated using ASTM (2017a) or AASHTO (2014a), of the same title. When water storage is required, open-graded aggregates with a minimum porosity of 30% are recommended. Many aggregate gradations typically used have higher porosities, often 38% to 45%.

Sieve analysis of washed gradations should be used per ASTM (2014b) or AASHTO (2014b), of the same title. ASTM (2017b) may be required to assess the percent smaller than (and passing) the 0.075 mm (No. 200) sieve. Subbase, base, bedding, and jointing aggregates typically use gradations in ASTM (2012a, 2016a) or AASHTO (2013a), which has the same gradations as the ASTM standards. Base materials are typically ASTM No. 57 stone with subbase aggregates meeting the requirements for ASTM No. 2, 3, or 4 gradations.

ASTM aggregate designations and gradation sizes are provided in Tables 4-2 and 4-3. Canadian Standards Association (CSA) grading requirements for coarse aggregate are provided in Tables 4-4 and 4-5. ASTM No. 2 stone would be similar in gradation to CSA Group II 80-40, and ASTM No. 57 to Group II 28-14, and ASTM No. 8 to Group II 5-2.5. Other similar gradations for the bedding, base, and subbase aggregates may be used supported by proven field performance in similar loads, soil subgrades, and climates (e.g., ASTM No. 89 bedding on an ASTM No. 67 base).

To help reduce settlement from one layer migrating into the layer below, aggregate layers should conform to the following choke criteria:

D_{50} Base/D_{50} Bedding layer < 25

D_{15} Base/D_{85} Bedding layer < 5

D_{50} Subbase/D_{50} Base < 25

D_{15} Subbase/D_{85} Base < 5

D_x is the particle size at which x percent of the particles are finer. For example, D_{15} is the particle size of the aggregate for which 15% of the particles are smaller and 85% are larger. This data point is obtained from an aggregate sieve analysis or by specifying the appropriate standard gradations as listed in Table 4-2 for metric units and in Table 4-3 for U.S. customary units.

4.2.5 Other Aggregate Base and Subbase Considerations

This standard allows for alternative aggregate gradations. When used in vehicular applications, they should meet the above mechanical requirements. Some local, state, and provincial transportation agencies specify well-graded, unstabilized aggregates as permeable bases (drainage layers) under conventional pavements. These aggregates can be used in PICP and can have a higher percentage of material passing the 4.75 mm (ASTM No. 4) sieve than those shown in the ASTM gradations.

These materials should have ≤ 2% passing the 0.075 mm (ASTM No. 200) sieve, and they typically require underdrains. Such gradations are sometimes used in arid areas or over weak or very low infiltration soils. These aggregates can provide desired structural capacity while providing some porosity for water storage and filtration.

Asphalt-treated (including porous asphalt) or cement-treated (including pervious concrete) open-graded bases can be used to increase the base stiffness and strength over weak soils or to support ESALs exceeding those described in this standard. Should the designer elect to use a bound base or subbase material, the potential for sedimentation or clogging of these layers should be considered. For the purposes of this standard, only designs with non-stabilized bases and subbases are covered. The use of stabilized bases or subbases used in PICP for ESAL applications over 1 million is limited and requires expertise of an experienced pavement engineer.

4.2.6 Concrete Paver, Aggregate Jointing, and Bedding Properties

This standard is based on the use of permeable interlocking concrete pavers. PICP pavers shall conform to ASTM (2016c) in the United States and CSA (2014b) in Canada. For vehicular traffic, pavers shall have an aspect ratio or length to thickness less than or equal to 3 and a minimum thickness of 80 mm (3 1/8 in.). Length and width dimensions do not include spacer bars or nibs, as shown in Fig. 4-2.

The joint widths for purposes of satisfying the stiffness parameters used in the design procedure should be 5 mm to 12.5 mm (3/16 to 1/2 in.). Joints should be completely filled with the jointing aggregate to the bottom of the paver chamfer. If no chamfers are present, the joints should be filled with aggregate to within 6 mm (1/4 in.) of the paver surface. ASTM No. 89 or 9 stone may be used to fill paver joints less than 10 mm (3/8 in.) wide.

The bedding layer should consist of a 50 mm (2 in.) thick layer of ASTM No. 8 stone screeded and leveled over the base layer. Metal or wooden rails are typically placed on top of the compacted base and are used to guide screeding elevations. The pavers should be placed immediately after the stone bedding is placed and screeded. Construction equipment and foot traffic should be kept off the screeded layer.

4.2.7 Concrete Paver Laying Patterns

The paver layer configuration pattern and stitching (when required) should be illustrated in project drawings and noted in the specifications. In addition, they should be included in the project method statement and quality control plan referenced in the guide construction specifications in Appendix B. Further guidance on laying patterns is provided in ICPI Tech Specs 11 and 15 (ICPI 2012a, b).

The layer configurations shown in this standard are samples of many available patterns. Other laying patterns may be used, as long as they have proven performance in similar load applications. Manufacturers of pavers used in PICP may have additional patterns, information, and test results that characterize their structural capacity and performance, as well as recommendations on using specific jointing and bedding aggregate materials. They may also have additional information that characterizes benefits of specific paver shapes on structural and hydrologic design, installation, and maintenance. Use of these patterns remains at the designer's discretion regarding general conformance to this standard.

The engineer may also consider using paving patterns suitable to machine installation. This equipment is illustrated in Fig. 4-3. The paving units are manufactured in the final or near-final laying pattern. The machine clamp grasps and places each layer, consisting of about 35 to 45 paving units at one time. Installation is substantially more efficient than manual placement.

A continuous 45° or 90° herringbone pattern should be used for vehicular pavements, as shown in Figs. 4-4a and 4-4b. As paving proceeds, a sailor (string) course is recommended against all straight raised or flush curbs, around all straight obstructions such as concrete collars, catch basins or drains, utility structures or boxes, building foundations, curbs, and adjacent pavements. This border course can contribute some stability under vehicular traffic while providing space next to raised curbs for the machine installation clamps. Adjacent curved edges, such as curbs, manholes, and some building foundations, may use soldier courses to better accommodate such curves. Sailor and soldier courses are typically installed manually. The presence of a full paving unit with chamfers next to a curb or concrete header reduces the risk of chipping and spalling compared to placing saw-cut pavers in such locations.

In some cases, the paver layer (cluster) configuration and the application determine the need for manually installing stitching pavers to create a continuous pattern. While this adds extra costs to mechanical installation and wasted half-size units, stitching contributes to interlock among clusters. This step helps reduce long, continuous joints, which reduces the risk of lateral instability under vehicular traffic. Figs. 4-5 and 4-6 illustrate herringbone paver layers jointed with stitching. Fig. 4-5 includes half-sized paver units removed and replaced with whole units. Fig. 4-6 allows half-sized paver units to remain.

Fig. 4-7a illustrates a 45° herringbone pattern that does not require stitching but creates a continuous laying pattern when installed. Because of a saw-toothed layer edge, extra care is required in packaging and transporting to prevent the layers and paving units from shifting. Edges of 45° paving patterns can be resolved with manufactured paving units, typically hand placed. An example is shown in Fig. 4-7b, called a "bishop's hat." Some paver clusters with indented sides of paving units mesh into each other; they also do not require stitching.

Fig. 4-8 illustrates how a layer pattern created by L-shaped pavers can be stitched together.

Load applications that include trucks require herringbone patterns or stitching clusters together. For lower vehicular load (cars only) or pedestrian applications, stitching may not be needed. It may be done only for aesthetic reasons. For such applications, half-sized pavers within herringbone patterns can remain in place and not be exchanged for whole pavers, as shown in Fig. 4-6. However, the larger layers should be offset when placed to eliminate joint lines longer than one layer. Figs. 4-9a and 4-9b illustrate other nonstitched laying patterns with offset layers.

4.2.8 Structural Behavior of the Aggregate Subbase

UCPRC conducted full-scale tests on PICP with dry and saturated aggregate subbases and developed mechanistic models to characterize rutting behavior up to 1 million ESALs. Eq. (4-5) was used to develop the UCPRC rutting model for the subbase aggregate layer:

$$RD_{SB} = aN^{b} \qquad (4\text{-}5)$$

where

RD_{SB} is the rut depth of the subbase layer;

N is the number of load repetitions; and

a and b are constants and are a function of the shear stress–strength ratio (SSR_{SB}) at the top of the subbase layer, calculated using Eqs. (4-6) through (4-9).

$$\text{Shear Stress–Strength Ratio } (SSR) = \frac{\tau_f}{\tau_{\max}} \qquad (4\text{-}6]$$

$$\tau_f = \frac{\sigma_1 - \sigma_3}{2}\cos\phi = \frac{\sigma_d}{2}\cos\phi \qquad (4\text{-}7)$$

$$\tau_{\max} = c + \sigma_f \tan\phi \qquad (4\text{-}8)$$

$$\sigma_f = \frac{\sigma_1 + \sigma_3}{2} - \frac{\sigma_1 - \sigma_3}{2}\sin\phi = \frac{\sigma_d + 2\sigma_3}{2} - \frac{\sigma_d}{2}\sin\phi \qquad (4\text{-}9)$$

where

$\tau_{\max}$ is applied shear stress acting on the failure plane oriented at an angle of 45° + $\phi/2$;

σ_f is applied normal stress acting on the failure plane oriented at an angle of 45° + $\phi/2$;

τ_f is the shear strength of the material under a certain stress state;

σ_1 and σ_3 are the major and minor principal stresses, respectively;

σ_d is the deviator stress, $\sigma_d = \sigma_1 - \sigma_3$;

c is the cohesion of the material; and

ϕ is the internal friction angle of the material ($\phi = 0$ for stress-independent materials).

A two-step model development process was followed:

- Step 1. Fit $RD_{SB} = a(dN + N_0)^b$ for each testing case with different subbase thickness (h_SB), test load (L), and test moisture condition (Dry and Wet), considering the effect of early embedment in the initial stages of trafficking. RD_{SB} is the total rut depth in the subbase for a load level i; dN is the incremental repetition under that load level i; N_0 is a model constant for considering the effect of earlier loading.
- Step 2. Fit $a \sim f(SSR_{SB})$ and $b \sim f(SSR_{SB})$ for all testing cases.

Using the rut test data from the load testing, the resulting rut depths showed an approximately linear relationship with load repetitions after early initial embedment for all testing cases. Consequently, the power constant b was set as 1. The constant a is a function of SSR_{SB}, calculated for each case using Eqs. (4-6) through (4-9).

4.2.9 Soil Subgrade Characterization and Behavior

The procedure for developing the rutting model for the subgrade was similar to that used for the subbase layer. The general formula for the rut model is the following:

$$RD_{SG} = aN^{b} \qquad (4\text{-}10)$$

where

RD_{SG} is the rut depth in the subgrade;

N is the number of load repetitions; and

a and b are constants and are a function of the shear stress–strength ratio (SSR_{SG}) at the top of the subgrade, calculated using Eqs. (4-6) through (4-9).

A two-step model development process similar to that described above was followed:

- Step 1. Fit $RD_{SG} = a(dN + N_0)^b$ for each testing case with different subbase thickness (h_SB), test load (L), and test moisture condition (Dry and Wet), considering the effect of early embedment. RD_{SG} is the total rut depth in the subgrade for a load level i; dN is the incremental repetition under that load level i; N_0 is a model constant for considering the effect of earlier loading.
- Step 2. Fit $a \sim f(SSR_{SG})$ and $b \sim f(SSR_{SG})$ for all testing cases.

Using the rut test data from the load testing, the rut depth in the subgrade had a power relationship of approximately 0.5 with load repetitions after early embedment (approximately 4 to 5 mm (~3/16 in.) for all testing cases. Consequently, the power constant c was set as 0.5. The constant a is a function of SSR_{SB}, calculated for each case using Eqs. (4-6) through (4-9). The subgrade rut model is summarized in Table 4-6.

4.2.10 Input Parameters for Mechanistic-Empirical Design of PICP

The default input parameters for mechanistic-empirical design of PICP are summarized in Table 4-7 and described in this section. The bedding and ASTM No. 57 base layers were combined with the pavers into a single surface layer, and the ASTM No. 2 subbase was analyzed as a separate layer.

Poisson’s ratio is assumed at 0.35. The default cohesion (c) of the subbase material remained the same at 0 kPa under dry and wet conditions. However, different default internal friction angles (ϕ) were assumed for dry and wet conditions. Selected values were 45° under dry conditions and 35° under wet, based on differences in rutting performance in this layer in the dry and wet tests and Kim and Tutumluer (2006) and Chow and Tutumluer (2014). The cohesion and internal friction angles of the subgrade vary with the resilient modulus ranging from 15 kPa (2.2 psi) to 25 kPa (3.6 psi) for cohesion, and from 15° to 25° for the internal friction angles.

4.2.11 Subbase Thickness Design Tables

Tables 4-8 and 4-9, indicating the No. 2 (or similar gradation) subbase thickness, present the metric and U.S. customary units. Designs allow rut depths up to 25 mm (1 in.). Higher rut depths may be acceptable in some facilities. All designs assume a 100 × 200 × 80 mm (4 × 8 × 3.125 in.) concrete paver, 50 mm (2 in.) ASTM No. 8 bedding layer, and a 100 mm (4 in.) thick ASTM No. 57 (or similar gradation) base layer. Designs should use subgrade resilient modulus based on wet conditions [(i.e., standing (detained) water in the subbase]. California bearing ratio (CBR) values are rounded upward after using Eq. (4-3). Local jurisdictions may adjust the minimum subbase thicknesses to suit local conditions or requirements.

The number of days in a year when the subbase has standing water in it can be conservatively estimated using Method 1 or Method 2, described as follows. An example using Method 1 is provided in Appendix A.

Method 1

1. Determine the contributing drainage area (CDA).
2. Determine the maximum number of hours of drawdown time (including the rainfall period). This is typically recommended by the provincial, state, or municipal stormwater agency. It is typically 24, 48, or 72 h.
3. Measure (or estimate) the soil infiltration rate on the project site and determine an average value.
4. Apply a 2X safety factor to the average value. Example: field measured average value = 2 mm/h (0.08 in./h); therefore, use 1 mm/h (0.04 in./h) for the design infiltration rate.
5. Calculate the daily infiltration depth for the soil, in other words design infiltration rate × 24 h, [(e.g., 2 mm/h × 24 h = 48 mm (1.89 in.)].
6. If there is a CDA, convert it to ratio by dividing the CDA area by the PICP area. Then adjust the daily infiltration depth for the soil by dividing it by the CDA ratio. Assume all CDAs as 100% impervious for estimating purposes. Example: Daily infiltration depth = 48 mm/CDA ratio of 2 = 24 mm (0.9 in.).
7. Use publicly available rainfall and/or climate normal for a meteorological station close to the project site or one that represents the regional climate [(i.e., Government of Canada website (http://climate.weather.gc.ca/historical_data/search_historic_data_e.html), U.S. National Oceanic and Atmospheric Administration (NOAA) (http://hdsc.nws.noaa.gov/hdsc/pfds/)] or others.

a. Select a data set that represents a typical yearly precipitation record, or, for added certainty;

b. Select a year that represents a wetter than average yearly precipitation record. For a more representative analysis, select multiple years and average those values.

8. From the collected data set(s), count the number of days that the rainfall exceeds 24 mm/day. If using multiple years, determine the minimum, maximum, and average days per year that the rainfall exceeds 24 mm/day. An example is provided below for a typical year in 2005, in which the number of days when the rainfall exceeds 24 mm/day is 9, as shown in Fig. 4-10.

9. Add the total number of days in the year that water has not completely infiltrated (or drained out via underdrains) and remains in the subbase. Use that total as an input to finding the subbase thickness in Tables 4-8 and 4-9.

Method 2

For jurisdictions with rainfall frequency spectrum graphical data,

1. Determine the contributing drainage area(CDA).
2. Determine the maximum number of hours of drawdown time (including the rainfall period). This number is typically recommended by the provincial, state, or municipal stormwater agency. It is typically 24, 48, or 72 h.
3. Measure (or estimate) the soil infiltration rate on the project site and determine an average value.
4. Apply a 2X safety factor to the average value. Example: field measured average value = 2 mm/h (0.8 in./h); therefore, use 1 mm/h (0.04 in./h) for the design infiltration rate.
5. Calculate the daily infiltration depth for the soil, in other words design infiltration rate × 24 h [(e.g., 2 mm/h × 24 h = 48 mm (1.89 in.)].
6. If there is a CDA, convert it to ratio to the PICP area. Then adjust the daily infiltration depth for the soil by dividing it by the CDA ratio. Assume all CDAs as 100% impervious for estimating purposes. Example: Daily infiltration depth = 48 mm/CDA ratio of 2 = 24 mm (0.9 in.).
7. From the rainfall frequency spectrum (RFS) graph, determine the percentile of rainfall events that exceeds 24 mm. Fig. 4-11 illustrates how to determine this percentile. To calculate the number of wet days, subtract the percentile of rainfall events that exceed 24 mm from 100% and multiply by 365 days/year. Example: (100% – 96%) × 365 = 14.6 days. Round up to 15 days. Use that total as an input to finding the subbase thickness in Tables 4-8 and 4-9.

4.2.12 Structural Design Life

A structural design example is provided in Appendix A, enabling the engineer to develop a pavement to meet the performance requirements (anticipated vehicular loads) over the design life, typically defined as ESALs or Caltrans traffic index. The structural design life (performance period) of a pavement is the time until the PICP is no longer able to satisfy the performance requirements. The end of structural design life is typically exhibited by rutting from wheel loads from shear failure and/or settlement. Structural rehabilitation typically addresses shear failure of the bedding, base, subbase, or subgrade soils. Beside rutting and settlement, the end of the structural design life can also be indicated by some damaged pavers, loss of jointing material, or edge restraint damage. In many cases, these distresses can be remedied by removing and replacing the damaged areas to cost-effectively extend the service life of the entire pavement.

4.2.13 Geosynthetics

The three primary types of geosynthetics used in PICP are geotextiles, geogrids, and geomembranes. Geotextiles should conform to subsurface drainage requirements in AASHTO (2015a). Geotextile strength properties should conform to Class 1 (highest strength) if exposed to severe installation conditions with greater potential for geotextile damage, (e.g., exposed bedrock, roots, or aggressive construction operations).

Geotextiles can be used to prevent the migration of fines into the aggregate layers, which causes a reduction in the structural capacity of the aggregate layers. Class 2 geotextiles are typically used in PICP to resist punctures or tears from dumping and compacting aggregate. Geotextile is typically not placed in interstitial open-graded aggregate layers, because it provides little to no increase in structural capacity. In addition, the presence of a horizontal geotextile layer at any elevation could risk clogging over time. There is currently little in situ forensic evidence for permeable pavements demonstrating long-term clogging of geotextiles or lack of it. For that reason, geotextile placed horizontally on the subgrade is specified at the designer’s discretion. Geotextile should be placed vertically against the walls of excavated soil for all applications that do not use a full-depth concrete curb to separate the PICP pavement base or subbase from adjacent soils.

Geogrids may also be used in PICP. They are often used as pavement reinforcement to improve the structural capacity of the subgrade by bridging the load in the pavement section above. Very low CBR soils may require the benefits of a geogrid in the pavement section. There is little research or design data to demonstrate substantive structural support when placed within the aggregate layers. Three-dimensional webs or cellular soil or aggregate confinement systems may also provide additional structural support, and manufacturer design recommendations should be followed.

Geomembranes may be used to separate the PICP from adjacent structures by providing a watertight barrier. Geomembranes can include polyvinyl chloride (PVC), ethylene propylene diene monomer (EPDM) similar to that used as single-ply waterproof roofing membrane, or high-density polyethylene (HDPE). Geomembranes used for permeable pavement applications should follow manufacturer's recommendations and typically have a minimum thickness of 0.762 mm (30 mil). Geomembranes typically require an overlay of nonwoven geotextile on both sides to protect them from the aggregates tearing or puncturing them during construction. They also typically require welded or glued joints for a watertight barrier.

4.2.14 Edge Restraints

All pavements should be designed with edge restraints (typically concrete curbs) at the perimeter. Edge restraints for vehicular applications are typically constructed using concrete curbing, a minimum of 150 mm (6 in.) wide and 225 mm (9 in.) deep, placed on top of the subbase layer or other base aggregate. Edge restraints help maintain rotational and horizontal interlock in the pavement surface resulting from dynamic vehicular wheel loads, such as turning, braking, and accelerating. Curb width and depth as well as overall stability should be evaluated in sizing edge restraints for heavy vehicle applications.

Pedestrian and occasional light vehicle applications may use alternative edge restraints (e.g., metal or plastic edging spiked in dense-graded aggregate shoulder berms or attached to geogrid if the supplier has demonstrated engineering data to support connection strength and load dispersion through analytical measurements). Guidance on edge restraints is provided in ICPI Tech Spec 18 (ICPI 2013).

4.2.15 Pedestrian-Only Use

In many applications, pedestrian areas receive some vehicular use. This use typically consists of service, construction events, or emergency vehicles. If the pavement is expected to receive any vehicular traffic, the structural design procedures outlined in Section 4.2 should be used to account for the dual purpose of the pavement.

Pedestrian-only applications can use 60 mm (2 3/8 in.) thick pavers and a 50 mm (2 in.) thick layer of high-quality aggregate bedding material (typically ASTM No. 8 stone) over the base. A minimum 150 mm (6 in.) thick base (typically ASTM No. 57 stone) is also typically recommended. A subbase is optional and typically consists of larger, open-graded aggregate placed under the base. A minimum 150 mm (6 in.) thick subbase is recommended when the soil subgrade strength is lower than a CBR of 3 (resilient modulus less than 36 MPa) or when additional water storage is required.

4.3 HYDROLOGIC ANALYSIS

A hydrologic analysis involves developing and applying a manual or computerized computational model to characterize the movement of water through the system, water infiltrating into the soil subgrade, and outflow through underdrains. In general, the hydrologic analysis assesses how the design water volumes or hydrographs can be infiltrated, stored, and released by the pavement structure provided. The quantity of water received and various paths taken to exit the pavement system are often described as a water balance.

The first step in the hydrologic analysis is to verify the hydrologic goals for the project. These goals may include, but are not limited to the following:

1. Stormwater volume control—maintain or reduce predevelopment stormwater volumes and/or peak flows;
2. Water quality—Meet minimum volume capture or sizing criteria (e.g., capture first 25 mm or 1 in. of rainfall or stormwater depth equivalent to 80–90% of the average annual runoff) over the contributing drainage area to accommodate first flush and/or contaminant removal efficiencies (e.g., 80% total suspended solids (TSS) removal);
3. Water thermal characteristics—maintain or reduce predevelopment temperature of surface or groundwater discharging to the receiving body;
4. Flood and/or peak flow control—retain or detain specific design storms (e.g., 100-year, 24 h duration storm event) to prevent downstream erosion or capacity limitations with existing stormwater infrastructure;
5. Downstream erosion control—eliminate or minimize downstream erosion potential by limiting critical discharge or reducing outflow volumes and flow durations;
6. Infiltration and/or recharge targets—maintain or increase groundwater recharge rates to prescribed targets or thresholds in order to maintain predevelopment water budget requirements; and
7. Ecosystem and habitat—maintain the existing hydrologic regime, including surface and groundwater interactions and shallow base flow necessary to maintain significant vegetation communities, wetlands, and aquatic habitat.

Consult with the local regulatory agency to determine which goals apply to the project site.

The second step is to assess the site conditions, including but not limited to quantifying the contributing drainage area (CDA), measuring the infiltration rate of the subgrade soils, and identifying potential receptors for the underdrains (when used). These inputs and outputs are discussed in greater detail in Section 4.3.1.

The third step is determining what type of PICP system will be used: full, partial, or no infiltration, as initially presented in Section 3.2, and discussed in greater detail in Section 4.3.2.

The fourth step is to complete the hydrologic design for the permeable pavement. There are many methodologies that can be used, depending on the project requirements and the desired accuracy of the results. Common methodologies include the following:

1. **Water balance estimate**. This method is used to determine if a permeable pavement is feasible for a site and to find the appropriate system for preliminary costing purposes. Section 4.3.3 provides details for developing water balance estimates.
2. **The time-step analysis**. This approach can be completed as a coarse estimate of the water budget using a limited number of time steps. This analysis method is more appropriate for full-infiltration designs with no CDA. For partial- or no-infiltration designs, this method (using a spreadsheet or other tool) may not have a sufficient number of time steps to develop inflow and outflow hydrographs distributed over the rainfall event and draw down time. In some cases, a detailed time-step analysis is required to better characterize concurrent inflows and outflows. Such analysis often includes run-on from CDAs and subgrade infiltration, as well as outflows from underdrains and other control devices, such as orifice or weir structures.
3. **Event-based hydrograph estimation methods**. These models generate an estimated hydrograph for a specified storm event. Examples include the Watershed Hydrology (WinTR-20) program, Small Watershed Hydrology (WinTR-55) program, Santa Barbara Unit Hydrograph (SBUH) program, HEC-1 Flood Hydrograph Package, HydroCAD Stormwater Modeling (HydroCAD), and ICPI Permeable Design Pro software, among others.
4. **Continuous simulation modeling methods**. These models generate long-term hydrographs from multiple storm events based on a continuous rainfall record and other hydrologic inputs. Many models also have the capability to route hydrographs through stormwater management facilities and conduct analysis of transient inflows, outflows, and storage levels using the designed elevation-based storage and discharge capabilities within the pavement system. Examples include the USEPA Stormwater Management Model (SWMM), the Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS), Computational Hydraulics International (CHI-PCSWMM), Source Loading and Management Model for Windows (WinSLAMM), and the Integrated Design Evaluation and Assessment of Loadings (IDEAL) model. Some of these models also accommodate event-based hydrograph estimation methods and can estimate the production and transport of pollutant loads associated with stormwater runoff.

Selection of the hydrologic design method can be based on the considerations and range of model applications given in Table 4-10.

Some regulatory agencies request analysis results using the rational method and the simple method. For permeable pavement design, the use of these methods is not recommended. If used, calculations should be checked against other design methods.

Some regulatory agencies request analysis results reported as a runoff coefficient (C) value based on the rational method. A runoff coefficient (C) represents the percentage of precipitation that converts directly to surface runoff for a particular surface type; it is used to approximate peak runoff rates to the receiving stormwater conveyance system. A permeable pavement is typically designed to either minimize surface runoff by storing and infiltrating water into the soil subgrade in the case of full- or partial-infiltration systems, or to store water for controlled discharge, as a detention pond does in the case of no-infiltration systems. Therefore, a traditional C value cannot be realistically applied.

Other agencies may request an NRCS curve number (CN) (NRCS 2004) for the PICP area per USDA Technical Release 55 (USDA 1986) entitled *Urban Hydrology for Small Watersheds* from the USDA Natural Resources Conservation Service (NRCS). A CN represents average antecedent runoff conditions from a site based on the infiltration characteristics of the drainage area as a function of soil type, land cover type(s), antecedent rainfall, impervious surfaces, and retention of rainfall by various surfaces.

For full- and partial-infiltration designs, discharge (via underdrains) typically only occurs after the infiltration capacity of the subgrade soil over the duration of the design storm and reservoir storage capacity below the outflow pipe are exceeded. The following adaptation of the NRCS runoff equation can be used to estimate the adjusted total runoff volume in depth (i.e., underdrain discharge) for full- and partial-infiltration systems. In the case of no-infiltration designs, an orifice equation or similar equation can be used to calculate the adjusted total runoff volume (Q); details on orifice equations are provided later in this standard.

The following equation from TR-55 (USDA 1986), is a simple method for determining the CN for permeable pavement:

$$Q_{\mathrm{adj}} = (P - I_a)^2/P - I_a + S \qquad (4\text{-}11)$$

where

Q_{adj} = Total runoff depth (mm or in.);

P = Total precipitation depth (mm or in.);

I_a = Initial abstraction (mm or in.) of losses, specifically subgrade infiltration over the duration of the design storm; and

S = Storage parameter (mm or in.), specifically the storage depth in the PICP base or subbase reservoir below the outflow for the underdrain.

Eq. (4-12) can then be used to calculate the CN_{adj} based on the resulting total rainfall depth:

$$\text{CN}_{\text{adj}} = \frac{1000}{10+5\text{P}+10 \text{ x } \text{Q}_{\text{adj}}-10 \text{ x } \left(\text{Q}_{\text{adj}}^{2}+1.25 \text{ x } \text{Q}_{\text{adj}} \text{ x P }\right)^{1/2}} \qquad (4.12)$$

For example, the 24 h design storm for a site is 200 mm (8 in.); $P = 200$. The native soils exhibit an infiltration rate of 2.5 mm/h (0.1 in./h) or 60 mm/day (2.4 in./day); $I_a = 60$. The underdrain of the PICP system is elevated to provide 60 mm (2.4 in.) of water storage before overflow begins; $S = 60$. The resulting total rainfall depth (Q_{adj}) using Eq. (4-11) is 98 mm (3.92 in.), and the corresponding CN_{adj} using Eq. (4-12) is 65.

Some caution should be exercised in applying the NRCS method to calculating runoff in catchments smaller than 2 ha (5 acres). The NRCS method is intended to calculate runoff from larger storms (2-, 10-, and 100-year return periods) with 24-h durations and from larger catchments or watersheds. Therefore, the NRCS procedure tends to underestimate runoff from smaller storms in small drainage areas.

4.3.1 Overview of the Hydrologic System

The main components of the water balance are presented in Fig. 4-12 and are described in detail in the following sections. Evaporation, wetting of materials, and lateral migration of groundwater into the storage reservoir are not considered in the water balance analysis because they are typically negligible compared to the other components.

4.3.1.1 Input—Direct Rainfall

Direct rainfall, as the title indicates, is a direct function of the precipitation that occurs at the site. Rainfall is quantified by the following characteristics:

- Depth (mm or in.)—Total amount of rainfall occurring during the storm event;
- Duration (h)—Length of time during which rainfall (storm event) occurs; and
- Intensity (mm or in. per h)—depth divided by the storm duration.

The design requirements for direct rainfall are typically supplied by the municipality or other regulatory agency responsible for stormwater drainage; they can often be expressed in terms of an exceedance probability or a return period. The exceedance probability is the probability that a storm event having the specified duration and depth will be exceeded in a given time period. The return period establishes the probability of a rain event with the same duration and depth occurring (e.g., a 10-year, 24 h storm event). Rainfall can be a single event or a continuous simulation of long-term rainfall records. They are used in the hydrologic design of the permeable pavement system.

4.3.1.2 Input—Contributing Drainage Area and/or Run-On

Many agency guidelines allow PICP systems to receive run-on from roofs, adjacent impervious pavement, and/or stabilized pervious areas (e.g., lawns), assuming that the system can accommodate the additional water. Run-on from unstabilized areas such as planter areas or gravel pavements are typically discouraged unless sediment control prevents contaminated water from entering the PICP system. High sediment loads lead to more frequent surface maintenance.

To quantify the run-on, the total area of each run-on source should be defined, and the contributing runoff from each source should be estimated using the design storm(s) as adjusted based on standard runoff estimating practices. Water run-on to the surface of PICP is typically modeled as sheet flow based on the applicable runoff coefficient (C) when using the rational method or Technical Release 55 (USDA 1986). Roof downspouts and other directly connected impervious areas are discharged directly into the base or subbase, or onto a splash pad before flowing onto the PICP area (to slow the velocity sufficiently to prevent washout of the joint material). When possible, roof downspouts should use filters to reduce sediment before entering the PICP. Designs should include pretreatment to reduce high sediment concentrations in run-on from contributing drainage areas directly connected to the PICP system.

4.3.1.3 Output—Subgrade Infiltration

The amount of water entering the subgrade is primarily determined by the subgrade permeability in full- and partial-infiltration designs. While the terms *soil infiltration* and *soil permeability* are used interchangeably in this standard, the former is the rate over time of water *entering* a soil's surface, and the latter is rate over time of water flow *through* the soil. For design purposes, the saturated permeability (conductivity) of soil requires estimation through in situ measurements. Values for measured saturated conductivity depend on the soil texture (particle size distribution) and structure (particle arrangement), especially density. Conductivity is also influenced by surface conditions where the measurement is taken, the permeability or hydraulic gradient of soil layers underneath, lateral flow(s), the extent of saturation within the soil subgrade profile, chemical and physical properties of the soil, pressure head, and stormwater temperature.

For preliminary site assessments, a local soil classification should be sufficient to provide an initial estimate of the saturated permeability or conductivity rate as well as to suggest the structural capacity of the subgrade soils. The USDA Natural Resources Conservation Service (NRCS) provides a

Web Soil Survey that identifies soil names at a site scale (http://websoilsurvey.nrcs.usda.gov). A hydrologic soil group (HSG) for each soil name is provided.

Table 4-11 can be used to estimate saturated hydraulic conductivity based on the NRCS hydrologic soul group. This table relates to uncompacted native soils. The rates in this table can be compared to permeability rates in the table (ASTM 2011a) that is Appendix E. The table is from the FHWA report, *Highway Subdrainage Design* (Moulton 1980) and is from values originally developed for earthen dam construction. Therefore, the permeability values given in Appendix E are conservative compared to Table 4-11.

For project designs, saturated soil conductivity or permeability should be determined by testing the subgrade soils in situ. Local jurisdictions may have specific test method requirements. When not locally specified, permeability testing methods should include use of a constant head well permeameter (i.e., Guelph Permeameter method), a constant head double-ring infiltrometer (e.g., ASTM 2009), a constant head pressure (single-ring) infiltrometer, or in situ falling head permeability using a lined borehole. Percolation tests often used for the design of septic drain fields with an unlined borehole or a test pit are not recommended, because they can overestimate soil conductivity rates. More accurate estimates require some restriction of lateral flow during testing to create one-dimensional vertical flow. Further details on conductivity testing methods are provided in ASTM (2016g). Regardless of the testing methodology selected, the practitioner should select a method that provides an accurate estimate of the in situ saturated hydraulic conductivity and infiltration rate.

The number of infiltration tests on a project site depends on factors such as the PICP area, subgrade soil type, current and past land use, and final subgrade elevation(s) from cuts or fills. The test locations should be selected to accurately characterize the infiltration capacity of the subgrade soils within the project area. In the absence of any site-specific information, geotechnical engineering recommendations, or local agency guidance, the recommended number of tests for determining the soil subgrade infiltration is a minimum of two tests up to 700 m^2 (7,000 ft^2) and one additional for each additional 700 m^2 (7,000 ft^2) of PICP. Measurements should be taken at the elevation of test pits to approximate the interface of aggregate subbase and the native soil (i.e., the subbase bottom elevation(s). The field-measured infiltration rate should be based on the average of all measured rates. Linear projects may require adjustment to determine the appropriate frequency of testing.

The tests should be conducted at the approximate elevation of the bottom of the pavement structure with soil logs recorded to at least 1 m (3 ft) further below that include a determination of the seasonal high groundwater table. Additional tests at various depths (soil horizons) may be required by the engineer in areas where soil types change, near rock outcroppings, in low-lying areas, or where the seasonal high water table is likely to be within 0.6 to 1 m (2 to 3 ft) from the surface. When stratified soil conditions exist, saturated hydraulic conductivity and infiltration rates used in design should be based on the most limiting soil layer and may include the application of an infiltrate rate reduction factor (i.e., safety factor), which considers the stratified soil conditions.

If the engineer deems that the soil subgrade requires compaction to meet structural requirements, the following procedure should be followed:

1. Obtain soil samples from the test pit(s) at the anticipated subgrade surface elevation and test samples to determine their laboratory Proctor density per ASTM (2012b) or AASHTO (2017b).
2. Conduct a soaked 96-h California bearing ratio (CBR) test on the soil sample(s) per ASTM (2016f).
3. Compact the soil at the bottom of the test pit(s) and ensure that the required density is being met at an optimum moisture content using a nuclear density gauge. Use the testing procedure in ASTM (2017e).
4. Conduct an infiltration test on the compacted soil at the bottom of the test pit(s). If there are no local requirements on test methods, use ASTM (2009) for soils with expected infiltrate rates between 10^{-2} and 10^{-6} cm/s (14 and 0.001 in./h).

Density and infiltration rate data help the engineer determine the extent to which compaction affects the infiltration rate. These data are used as input to the hydrologic design. The soaked CBR data (and related laboratory density data) are input to determine the aggregate subbase thickness. CBR is converted to resilient modulus as design input for determining the subbase thickness in Tables 4-8 and 4-9.

Once the in situ infiltration rate is determined, typically an infiltration reduction factor of safety is applied, as determined by the engineer if not required by the local approval agency. By applying the reduction safety factor, the design infiltration rate is determined. The infiltration reduction factor accounts for the following:

1. Particulate that passes through the jointing material or is washed off of the base or subbase aggregates and deposited on top of the subgrade surface. For fine-grained subgrade, such as silts and clays, which have generally low permeability, the effect of the deposited material on infiltration is minimal. For coarse-grained subgrades, such as sand, the deposited material may significantly reduce its permeability. Infiltration into the sides of the excavation (if no full-depth curbs or geomembrane) may mitigate this condition.
2. The potential loss of permeability caused by soil compaction during construction (above the specified compaction).
3. The potential loss of permeability caused by subgrade consolidation from traffic loading and resulting rutting.
4. Significant variability in the in situ soil properties on the site.

5. Stratified soil conditions where an infiltration-limiting soil layer exists.

As the depth of stored water increases, the static head pressure is expected to increase, which can directly affect the infiltration rate. Some models provide more sophisticated algorithms for simulating infiltration into the subgrade. They account for the initial unsaturated state of the subgrade material and simulate higher initial infiltration rates at the beginning of an event, declining with cumulative time or infiltrated volume. However, given the uncertainty of infiltration rate estimates, and because saturated infiltration tends to occur mostly during significant storm events, more sophisticated analytical methods assessing static pressure in resulting soil infiltration rates are not generally necessary.

Where the seasonably high water table is within 0.6 m (2 ft) of the bottom of the proposed system, it may be necessary to conduct a recovery analysis to account for mounding of groundwater beneath the system. Mounding occurs if the unsaturated soil below the pavement becomes saturated and further vertical movement of water is prevented; as a result, water exiting the system begins to flow horizontally. Under these conditions, infiltration measurements obtained through a double ring or similar test are not valid, and additional testing to determine the saturated horizontal hydraulic conductivity of the soils is required. There are several methodologies available for modeling horizontal saturated flow and its applicability to a site; the potential for groundwater mounding should be investigated by the engineer. Subgrade infiltration rates tend to be low when a seasonal high water table is encountered. To minimize potential risk to groundwater, permeable pavement system designers are encouraged to contact their local water quality or supply agency to identify any required separation from local groundwater supplies.

4.3.1.4 Output—Underdrain Discharge

With partial- and no-infiltration systems, one or more underdrains are located at the lower elevation(s) of the system to allow for controlled discharge to a receiving water body or stormwater conveyance system. The underdrain discharge flow rate is a function of the hydrologic design goals for the project. This design goal can be matching the stormwater regulatory agency’s definition of predevelopment hydrologic conditions or other goals. Examples of agency requirements for detention of stormwater include the following:

- Minimum drawdown time (e.g., 24 h) to ensure adequate retention time for water quality treatment;
- Maximum drawdown time (e.g., 48 to 72 h) to allow the subgrade to dry before a subsequent storm event and prevent stagnant water associated with longer drawdown times; and
- Maximum permissible peak discharge (release rates) to protect downstream aquatic resources, to provide downstream flood protection, or to comply with stormwater conveyance system capacity limitations.

The common ways of controlling the rate and volume of water discharged through the underdrain(s) include one or more of the following:

1. Using upturned elbows or raised outlets at the end of the underdrain to create storage volume below the outlet invert;
2. Constructing an outlet structure with a weir (Fig. 4-13) to create storage volumes below the weir invert; or
3. Restricting the diameter of the underdrain, or adding an orifice plate or other flow restrictor, to regulate the discharge flow rate.

Examples of orifice and common weir equations are included in Appendix C.

For full- and partial-infiltration systems, any water below the outlet invert is expected to infiltrate. The detention time for completing infiltration is typically 24 to 72 h, or as specified by local regulations. The infiltrated volume during this holding time can help satisfy volume reduction, groundwater recharge, and/or water balance targets. In cases with detention times exceeding 24 h, the detained water can also become anaerobic, which can contribute to denitrification and to other pollutant reductions. Hydrologic and geotechnical expertise is required for the design of systems with detained water.

For no-infiltration systems, detained water can be reused for irrigation or in building graywater systems. Detained water design requires a reservoir sump with a submersible pump and an automated control system to remove the volume remaining from the preceding rainfall event and create sufficient volume to detain the water volume from the next event. The detained water can be reused for irrigation or in building graywater systems.

For designs that must comply with a minimum or maximum drawdown time, an outlet control device similar to that shown in Fig. 4-13, or an observation well (see Fig. 4-21), should be used that permits accurate water level monitoring within the reservoir.

4.3.1.5 Output—Surface Overflow

The design of all permeable pavement systems, like all stormwater designs, must include a method for safely conveying overflows of the system during extreme events. There are two situations that can result in overflow:

1. The rainfall event intensity (often with run-on) overwhelms the surface infiltration capacity, causing surface runoff from clogged PICP jointing material. Surface infiltration capacities typically range between 7,000 mm/h (276 in./h) and 20,000 mm/h (787 in./h) for new PICP surfaces. Limited or no surface cleaning can result in sufficiently low surface infiltration, which generates runoff.

2. Overflow of the storage volume within the pavement structure. This means of managing overflows is not recommended, because overflows emerging from the PICP surface can mobilize and transfer sediment in the jointing material into the storm drainage system.

While full-infiltration systems are designed to infiltrate the entire rainfall volumes, a safe overflow conveyance or bypass for high-intensity storm events that may exceed the capacity of the system should be included in full-, partial-, and no-infiltration designs. The overflows can be directed to a drainage structure using the following methods:

- Underdrains in the base or subbase connected to a manhole structure with a weir or other design controlling outflow;
- A catch basin placed immediately next to the pavement base or subbase with an opening in the basin side to directly receive overflows from the base or subbase; or
- A surface flow bypass via a curb cut to bioswale, a stabilized channel, or other conveyance structure.

4.3.2 Selection of the PICP System Type

PICP system types are defined by the degree of infiltration that occurs into the native subgrade, which is a function of the infiltration capacity of the soils. The selection process for choosing a hydrologic design type is summarized in Fig. 4-14.

4.3.2.1 No-Infiltration System

No-infiltration systems are most common in areas where the subgrade layer permeability is negligible, or where water capture and reuse is desired. No-infiltration systems can also be used

- Adjacent to building foundations,
- For protection of underground utilities,
- Where wellhead or groundwater protection regulations apply,
- Where conflicts with nearby septic systems are anticipated,
- At brownfield sites to minimize contamination migration,
- Where bedrock and or water table separation is not possible,
- In karst geologic conditions, and
- In areas of poor subgrade support where water adversely affects the structural capacity of the subgrade or causes undesired expansion of the subgrade.

No-infiltration systems are typically designed with an underlying geomembrane that prevents water from entering the subgrade, as shown in Fig. 4-15. The water is detained within the open-graded aggregates until such time as it is discharged under controlled conditions (based on the discharge limits) through the underdrain. In essence, this design type creates a retention pond filled with rocks that also serves as the pavement base or subbase. The hydraulic design is therefore the same as that used for detention ponds, where inflow rate and volumes are balanced against storage and timing of outflow rates and volumes. Therefore, the engineer may elect to use conventional detention pond design and modeling methods for no-infiltration designs.

Local agency requirements should be consulted to verify the design requirements (e.g., maximum allowable drawdown time or underdrain discharge limits) for the permeable pavement system. Typically, the maximum allowable drawdown time is specified by provincial, state, or local design regulations to ensure that

1. The full storage capacity of the reservoir is available before the next rainfall event, and
2. During winter months in northern climates, the water infiltrates the subgrade or drains the reservoir to levels below the local frost line.

4.3.2.2 Full-Infiltration System

In areas where the subgrade has high infiltration rates (i.e., subgrade infiltration is near or above the rate at which water is introduced to the system, including any direct precipitation, run-on, and underdrain sources), a full-infiltration design can be used, as shown in Fig. 4-16. Although there is storage capacity within the base and subbase aggregates, it is expected that minimal water will accumulate in the base or subbase and therefore, the pavement system may be designed without underdrains at the discretion of the designer.

In areas where the subgrade has moderate infiltration rates (i.e., where subgrade infiltration is below the rate at which water is introduced to the system, including any direct precipitation, run-on, and underdrain sources, but does permit the full infiltration of all event volume during the specified maximum allowable post-rainfall storage time), a full-infiltration design can be used as shown in Fig. 4-16. It is the designer's responsibility to calculate the required drawdown time and the required storage volume.

4.3.2.3 Partial-Infiltration System

In areas where the subgrade soils have moderate to low infiltration rates, underdrains are used to control the amount of water retained within the system for infiltration with the balance being discharged to an outlet (underdrain)—hence, the name partial-infiltration system. For low-intensity storms, all water typically remains in the reservoir until it infiltrates to the subgrade. Only during higher intensity storms, when the water level in the reservoir exceeds the invert controlled by the underdrain outlet, does stormwater discharge occur from the system. Underdrain(s) are recommended at or below the bottom of the aggregate reservoir system to protect it during installation and from vehicular damage in use, as shown in Fig. 4-17. This assembly includes an elevated outflow pipe(s) or outlet control (flow restrictor) at the discharge point. Some jurisdictions specifically require underdrains for over low-infiltration subgrades or to protect against a loss of infiltrative capacity in the subgrade over time.

Partial-infiltration system designs balance subgrade infiltration rates, the maximum allowable storage (drawdown) time, and the underdrain discharge limits against inputs from direct precipitation, run-on, and underdrain sources to determine the required aggregate storage volume. Ideally, the designer should conduct a hydrologic analysis, which consists of tracking the inflows, outflows, and change in storage volume over sequential time steps through a simulation period. Throughout the analysis period, water enters and leaves the system with the difference between inflow and outflow rates reflected in a change in storage volume. This type of storage routing is conducted for a single event in much the same manner that it would be conducted for a multiple event or continuous simulation.

A variety of methods and tools are available for storage routing. For single-event simulation, routing can be conducted in a spreadsheet using the storage indication method (see FHWA [2002] for more guidance) or other simple approaches. For multiple-event or continuous simulations, more complex models can be used. Except in certain special conditions, most models or methods can provide a reliable representation of storage routing for permeable pavements.

If there are flood control or peak discharge control requirements, the use of a control structure, as discussed previously, facilitates accurate discharge calculations, which may be required for permitting and approvals. These control structures can be added to all types of PICP designs.

4.3.3 Water Balance Calculations

This section outlines how to conduct a simple volumetric estimation to help assess which type of system would be most applicable based on-site conditions.

4.3.3.1 Inputs

Input design parameters and outputs used in the hydrologic design for permeable pavements are graphically represented in Fig. 4-18, including definitions. Table 4-12 summarizes inputs and outputs by infiltration type.

4.3.3.2 Preliminary Estimate of Required Storage Volume

A preliminary sizing analysis is recommended as a first step to estimate the storage volume required of the permeable pavement system to meet the hydrologic design goals. Based on the results, more comprehensive analyses can be completed to fine-tune the design. This result can be accomplished by using simplified assumptions before determining the need for more advanced analysis. The following assumptions and their limitations are described as follows:

1. **Rainfall depth does not vary with time.** The design storm rainfall depth (P) is applied over the catchment area to generate a total volume of water. This situation differs from a time-step or continuous analysis, where rainfall rates vary over the duration of the storm as expressed through hyetographs. For example, NRCS rainfall synthetic distribution patterns (Types I, IA, II, and III) have a focused rainfall period occurring near the midpoint of the storm event, which can cause the maximum storage volume within the base or subbase to occur at a time before the end of the storm event (T_S).
2. **Run-on rates from contributing areas do not vary with time.** Run-on rates are constant over the storm duration and do not extend beyond the duration of the storm event (T_S). Time-step analysis would be required to model time of concentration fluctuations in run-on, while poststorm event run-on could cause the maximum storage volume within the base or subbase to occur at a time beyond the end of the storm event (T_S).
3. **Infiltration rates are constant.** A more comprehensive analysis would be required to adjust infiltration rates based on increased, then decreased, head conditions as water accumulates in, and drains from, the base or subbase and into the soil subgrade.
4. **Underdrains go instantaneously from no flow to full pipe flow conditions**. Partial pipe flow conditions are neglected.
5. **The outlet flow rate through the underdrains (Q_U) is constant.** A more comprehensive analysis would be required to adjust outflow rates for increasing, then decreasing, head conditions.
6. **The permeable pavement area is known.** The permeable pavement area is defined as the total permeable surface with a constant base or subbase thickness. In scenarios where base depth limitations exist (e.g., high water table, presence of utility devices, or shallow bedrock), an iterative process would need to be performed to balance V_W and A_p.
7. **The subgrade is level.** In situations where the subgrade is sloped, the storage volume of water (V_W) would need to be adjusted accordingly. Dealing with slopes is covered in more detail in Section 4.5.2.

Water balance equation $PA_p + RA_c - I(T_D + T_S)A_I - Q_U(T_D + T_S)Z$

where PA_p = Rainfall volume falling directly on the permeable pavement;
RA_c = Run-on volume from the adjacent contributing area;
$I(T_D + T_S)A_I$ = Infiltration volume into the underlying soil; and
$Q_U(T_D + T_S)Z$ = Outflow volume through the underdrains.

To ensure that the system is accommodating maximum water storage requirements, the volume of water that can be stored in the pavement base (V_W) needs to be based on conditions immediately at the end of the design storm (T_S), because there has been no opportunity for poststorm drawdown ($T_D = 0$). The associated hydraulic depth, which is the depth of water within the base or subbase when $T_D = 0$, can be calculated using Eqs. (4-13) and (4-14).

$$\text{Hydraulic depth} = \frac{V_W}{A_p * n} \qquad (4\text{-}13)$$

with

$$V_W = P(A_P) + R(A_C) - I(T_S)A_I - Q(T_S)Z \qquad (4\text{-}14)$$

Details on how to design each permeable pavement are provided below with worked examples in Appendix A.

4.3.3.3 No-Infiltration System Design

With no-infiltration system designs, there is no subgrade infiltration ($I = 0$). Also, $Z = 1$; adjustment for the elevation of the pipe outlet is not required because the underdrain is at the bottom of the pavement. Eq. (4-14) is further simplified to the following:

$$V_W = P(A_p) + R(A_C) - Q_U T_S \qquad (4\text{-}15)$$

As noted previously, the outlet flow rate through the underdrains (Q_U) needs to be a constant to conduct a preliminary sizing analysis. However, using the maximum flow rate out of the pipe for Q_U, whether it is based on an arbitrarily set discharge pipe diameter or some regulated pipe flow per the local stormwater criteria (Q_{reg}), which is controlled by an orifice or similar device, would overestimate the actual pipe discharge in all but extreme conditions. A representative percentage of the maximum flow rate should be used to estimate Q_U based on engineering judgment. The following example illustrates how to estimate a representative Q_U when using an orifice on the discharge pipe to maintain a regulated pipe flow:

1. When using the orifice equation (Eq. [4-16]), the flow rate through the pipe outlet (orifice) is a function of the driving head (H_O) above the centroid of the orifice (see Fig. 4-19 for a graphical representation with more details on the actual calculations in Appendix C). It is important to note that
    a. The driving head has to account for the void displacement (i.e., porosity) of stored water within the base or subbase aggregates; and

b. Where the underdrain is installed in a trench just below the subbase, the driving head equals the water depth within the base or subbase (hydraulic depth) plus the radius of the underdrain pipe (r_{pipe}) and any pipe cover between the subbase and the top of the pipe.

$$Q_U = C_D \times A \times \sqrt{2g\Delta H_O} \qquad (4\text{-}16)$$

where

A = Area of the pipe outlet;

g = Gravitational acceleration;

C_D = Coefficient of discharge; and

$$H_O = \text{ Hydraulic depth} + r_{\text{pipe}} + \text{pipe cover} \qquad (4\text{-}17)$$

2. Regulated pipe flows are based on worst-case conditions (i.e., when the water level would reach its highest point); to avoid a circular calculation between hydraulic head and pipe flow, the approach is simplified to determine the highest potential driving head ($H_{o\text{–max}}$) when there is no pipe discharge ($Q_U = 0$). Combining Eqs. (4-15), (4-16), and (4-17), and applying $Q_U = 0$, results in the following:

$$H_{O-\max} = \frac{V_W}{A_P{*}n} + r_{\text{pipe}} + \text{pipe cover} = \frac{(PA_P+RA_C)}{A_P{*}n} + r_{\text{pipe}} + \text{pipe cover} \qquad (4\text{-}18)$$

3. Since the radius of the outlet pipe (r_{pipe}) is intrinsic to the maximum storage head [Eq. (4-17)] and the area of the pipe within the orifice equation [Eq. (4-16)], it is necessary to conduct an iterative process to determine r_{pipe}. Start with an assumed r_{pipe} (50 mm (2 in.), as an example) in Eq. (4-17), then use the resulting maximum potential storage depth ($H_{o\text{–max}}$) and the maximum allowable discharge rate (Q_{reg}) in the orifice equation to calculate the cross-sectional area (and radius) of the pipe. Repeat this step as required until satisfied with the accuracy of r_{pipe}.
4. Set the design H_O to a factor of $H_{O\text{–max}}$ that represents common pipe flow conditions. Depending on how conservative an approach is taken, the design H_o can range between the radius of the pipe (r_{pipe}) and 1/3 $H_{O\text{–max}}$. Use this amount within the orifice equation to compute the flow rate through the pipe to be used in the water balance (Q_U).

Once the design pipe flow (Q_U) is known, V_W can be calculated using Eq. (4-15), then the hydraulic depth using Eq. (4-13).

4.3.3.4 Full-Infiltration System Design

With full-infiltration system designs, there is no discharge pipe used ($Q_U = 0$). To calculate V_W, Eq. (4-14) is further simplified to

$$V_W = P(A_P) + R(A_C) - I(T_S)A_I \tag{4-19}$$

It may be possible that V_W will be less than or equal to zero, particularly over soils with high infiltration rates, in which case the pavement thickness required for structural capacity will govern the design.

When V_W is greater than zero, then a second calculation is required to ensure that the subgrade does not remain saturated for longer than the maximum allowable post-rainfall storage time (T_D). The following equation is used to verify compliance:

$$T_D \geq \frac{V_W}{A_I * I} \tag{4-20}$$

When this condition is met, a full-infiltration design is acceptable. When this condition is not met, then a partial-infiltration design is required.

4.3.3.5 Partial-Infiltration System Design

With partial-infiltration system designs, the underdrain discharge (Q_U) and Infiltration (I) are present, so Eq. (4-14) is used without any simplification.

The volume of water below the invert of the pipe (or outlet structure) is referred to as *detention storage* and is regulated by the amount of water that can infiltrate to the subgrade within the maximum allowable post-rainfall storage time (T_D). Eq. (4-21) is used to calculate the pipe elevation (PE) required to provide the infiltration storage depth:

$$\mathrm{PE} = \frac{I * T_D}{n} \tag{4-21}$$

Note that Eq. (4-21) calculates how long it takes the infiltration storage volume to drain: engineering judgment is required to adjust T_D to account for the portion of the allowable post-rainfall storage time that water is expected to be above the pipe elevation (i.e., when pipe discharge is occurring). There is no simplified recommendation because the reductions in T_D can be anywhere between 10 and 90%, where a higher percentage reduction equates to a longer period of time that pipe flow occurs. Fortunately, whatever assumed value is used can be verified at the end of the calculations, and adjustments can be made if required.

The underdrain elevation factor (Z) is applied when the outlet pipe is raised above the subgrade, and it represents the percentage of time that underdrain outflow occurs. To calculate Z, two variables are used: T = time to fill-infiltration storage zone; and, V_{PE} = volume of water in the infiltration storage zone.

$$V_{\mathrm{PE}} = \mathrm{PE}\ \mathrm{x}\ A_P\ \mathrm{x}n = \frac{T}{T_S}\left(PA_P + RA_C - IT_SA_I\right) \tag{4-22}$$

Rearranging the formula provides

$$T = \frac{\mathrm{PE\ x}\, A_P * T_S\, x\, n}{PA_P + RA_C - IT_SA_I} \qquad (4\text{-}23]$$

and

$$Z = 1 - \frac{T}{T_S} \qquad (4\text{-}24]$$

The underdrain discharge rate (Q_U) calculation for partial-infiltration designs is similar to the no-infiltration system design, with the following adjustments:

1. Determine the highest potential driving head ($H_{O\text{–max}}$) to be used in the orifice equation by neglecting water discharge during the storm event. Allowing for infiltration, calculate V_W when $Q_U = 0$, then divide by the pavement area (A_P). In this case, because the underdrain invert is elevated, the infiltration storage depth (PE) and pipe radius (r_{pipe}) are subtracted to determine the driving head above the centroid of the discharge point.

$$H_{o-\max} = \frac{V_W}{A_P x\, n} - \mathrm{PE} - r_{\text{pipe}} = \frac{(PA_P + RA_C - IT_SA_P)}{A_P x\, n} - \mathrm{PE} - r_{\text{pipe}} \qquad (4\text{-}25)$$

2. Use the maximum allowable discharge rate and the maximum H_o to calculate the diameter of the underdrain pipe or size of the orifice used (Eq. [4-16]).
3. Set the design H_o to the radius of the pipe (which is the elevation above the centroid of the orifice area). Now that the orifice size is known, this result generates a conservative value for Q_U that neglects water buildup above the pipe.

Once the initial calculations are complete, check that the reduction in T_D above is appropriate. Adjust the T_D value in Eq. (4-21) to optimize the infiltration volume, which is governed by the pipe elevation.

Worked examples for each permeable pavement strategy are provided in Appendix A.

4.4 BALANCING STRUCTURAL AND HYDROLOGIC DESIGNS

When the hydrologic and structural design thicknesses are relatively equivalent, then the thicker pavement section should be used and the design is considered cost efficient. If the base and/or subbase thickness required for hydrologic design is much thicker than that required for structural capacity, the designer may choose to modify some of the design parameters to make the design more cost-effective. This modification may include the following:

- Changing the base or subbase aggregates to material(s) with greater porosity to permit greater storage in a smaller cross section.
- Increasing the frequency, diameter, or slope of outlet pipes to increase water outflow and decrease the required storage (if in compliance with regulatory requirements).

- Incorporating other stormwater storage systems or downstream stormwater management (treatment train) practices.

If the base or subbase thickness required for structural design is significantly thicker than that required for hydrologic design, the designer must increase the structural capacity while reducing the pavement subbase thickness, or simply reduce the pavement subbase thickness to accept a lower design life (defined by ESALs or TIs). Structural improvements that can reduce the overall pavement thickness may include the following:

- Increasing the thickness of the surface layer (i.e., the concrete pavers). These layers have a higher structural capacity than the base and subbase layers. Slight increases in the thickness of this layer may provide the necessary structural capacity improvement.
- Improving the quality of the base or subbase layers to provide a stiffer material (i.e., a higher resilient modulus).
- Using additional stabilized layers below the paver surface or applied onto the subgrade, such as asphalt or cement-stabilized, open-graded drainage layers that do not compromise permeability.
- For no-infiltration or partial-infiltration systems, stabilizing the subgrade soils below the liner. This stiffening is typically done with cement or lime mixed into the soil.
- Using stabilizing geosynthetic materials within or below the subbase.

A thicker subbase may result in excess capacity for hydrologic performance. The designer can choose to accept this surplus as a factor of safety against long-term subgrade clogging. However, it may also be possible to accept a higher degree of compaction of the subgrade, which would reduce the infiltration rate but strengthen the base material, thereby moving the structure to a more balanced and cost-efficient design.

4.5 ADDITIONAL DESIGN CONSIDERATIONS

4.5.1 Subgrade Scarification

Some permeable pavement design guidance documents require that the subgrade be ripped or scarified to increase infiltration before the placement of the aggregate base or subbase. Although this practice may increase the total surface available for infiltration, it has no effect on the permeability rate of the underlying soil. With disturbed soil underneath, the constructed pavement above may also be more prone to settlement or rutting under traffic. A geotechnical or pavement engineer should be consulted to determine the effect of subgrade ripping or scarification on the long-term performance of the pavement because these practices can increase the risk of settlement of the entire structure.

4.5.2 Slopes

Permeable pavements can be installed for sloped surfaces. The design of sloped surfaces should consider decreased constructability, decreased surface infiltration, and compliance with allowable pavement slopes for roads, parking lots, and other facilities. Subgrades and/or subbases can be designed with flat or sloped bottoms that do not need to match the surface slopes. Terracing of the subgrade with flat bottoms on each terrace helps reduce the overall stone depth and maximize infiltration. Subgrades and/or subbases can also be bermed and piped to control downslope flows and encourage infiltration through using check dams. These check dams may be required on subgrade slopes exceeding 2%, and they are recommended for slopes exceeding 5%.

The design for sloped systems should ensure that water is not able to surge from below and exit the surface of the pavement. The pavement structural design for sloped systems should ensure that the full structural section is present above the top of the check dam elevation, or that the check dams provide an equal amount of structural support for the pavement. Overflows for larger storm events should also be accounted for in the check dam design.

Flows can be slowed with check dams, allowing some filtering through the aggregate base or subbase before discharge through piping systems. The check dams can be made of concrete, geomembrane, geotextile-wrapped aggregate, or transverse trenches excavated into the subgrade. Slowing the flow through the geomembrane barriers via drainpipes is also recommended. An example of a water flow geomembrane barrier is shown in Fig. 4-20. Use caution with earthen berms used as check dams built within the base or subbase layers. If these structures are installed above the frost line, they can cause heaving of the pavement above. Similarly, consideration for the differential settlement over earthen check dams should be part of all designs.

4.5.3 Underground Utility Treatment

Minimum horizontal and vertical separation for underground utilities are commonly specified to protect lines from damage while an adjacent utility is being repaired and/or to prevent potential cross contamination (in the case of water and sewer lines). Whenever possible, PICP should be designed in accordance with minimum setbacks and separation requirements for utilities, as determined by the local agency or utility. In cases where this agreement is impossible, utility lines may require relocation outside of the PICP footprint. Otherwise, electrical and cable lines should be encased in waterproof conduit, and all other lines should be covered with an impermeable liner to prevent contact with infiltrating water.

When such utilities laterally cross through the permeable pavement base or subbase, a trench collar made of geomembrane material should be installed vertically against the subgrade around the utility line as it enters from the subgrade and exits the aggregate base or subbase returning into the subgrade. A liner helps prevent water from migrating laterally into the subgrade along the utility pipe. The liner should be sealed against the utility line and extend a minimum of 300 mm (12 in.) from around the utility line while in direct (vertical) contact with the subgrade.

The engineer should design levels of protection appropriate for utility lines penetrating a PICP base or subbase subject to vehicular traffic. This goal may be accomplished by encasing the utility line with concrete or controlled low-strength material (CLSM). Pervious concrete may be used to encase utilities if horizontal water flow through the PICP base or subbase is required and concrete or CLSM around the utility would cause an unacceptable blockage of the required flow rate.

PICP can be advantageous to utility maintenance requirements since the wearing surface can be reinstalled without saw cutting and patching of the utility trench. Unlike monolithic permeable surfaces, the absence of cutting does not reduce the life of the reinstated paver surface. For maintenance of underground utilities, they can be accessed using conventional excavation techniques. The first step is removing (and preserving) the pavers from the surface; the exposed edges of the surrounding undisturbed pavers should be restrained from movement toward the utility cut by providing a setback from the planned utility trench of at least 0.6 m (2 ft). The bedding, base, subbase, and underlying soils can then be excavated to expose the utility line.

While the bedding layer generally cannot be reused, excavated base and subbase materials may be separated so that they can be reused. A geotextile may be applied under any aggregate stockpiles set on permeable pavers, exposed soil, or grassed areas. Backfill materials such as CLSM, concrete, or dense-graded aggregates also can be used for reinstating small areas of excavated, disturbed soil if the configuration of such areas makes compaction difficult or impossible. Care should be taken to try to minimize the area of excavation in the reservoir layer since open-graded aggregates tend to slough into the trench. Once the utility repair is completed, and the subgrade, subbase, base, and bedding materials have been replaced and compacted, the pavers can be reinstalled and compacted following the original laying pattern. The joints are then filled with jointing material, and the pavers are swept clean and compacted again.

4.5.4 Monitoring Well

For large-scale applications and demonstration projects, PICP should include a 100 to 150 mm (4 to 6 in.) diameter vertical perforated pipe that serves as an observation well to monitor water level in the reservoir during or after storms. This information can be used to assess potential changes to the subgrade infiltration rate and/or clogging of the underdrain. Example details for monitoring wells are shown in Figs. 4-21 and 4-22.

The bottom of the pipe can penetrate the subgrade and should be protected and supported during base or subbase filling and compaction. The monitoring well pipe should be located at or near the permeable pavement system low point but at least 1 m (3 ft) away from the edge of the permeable pavement. The bottom of the pipe should be inserted into the subgrade because this step assists base or subbase filling around it. The well needs to be protected from damage during base compaction. The pipe should have a lockable cap to prevent vandalism that is removable to observe the water level during or after storms to determine the infiltration rate. The depth to the soil subgrade should be marked on the inside of the cap. Monitoring wells can be connected to the horizontal underdrain outflow pipe and can be used as a cleanout should sediment settle within the pipe.

4.5.5 Adjacent Buildings and Pavement Systems

Permeable pavements may be constructed adjacent to buildings or conventional pavements with dense-graded bases, provided that certain precautions are taken. Building foundations should be protected from water infiltration by sloping the permeable pavement away from the building and protecting the building foundation by waterproofing or installing an impermeable liner vertically against the foundation wall and along the nearest sides and bottom of the permeable pavement. All buildings within 3 m (10 ft) of the PICP must be protected. An example showing the use of an impermeable liner to protect a building foundation is provided in Fig. 4-23.

Adjacent conventional pavement bases, subbases, and subgrades are not designed for saturation and therefore require protection from water infiltration. This protection can be achieved by separating the two pavement systems using impermeable barriers such as geomembranes or concrete. Consideration should also be given to sloping the permeable pavement subgrade away from adjacent conventional pavements and structures. Another option is providing underdrains within the permeable pavement base at the interface of the conventional pavement base. Example transition details for flexible and rigid pavements are shown in Figs. 4-24 and 4-25.

4.6 WATER QUALITY BENEFITS

In the urban environment, frequent, smaller storms generate the majority of stormwater pollution. For this reason, many jurisdictions include a water quality design storm or criteria (e.g., capture and treatment of first flush volume, annual TSS removal). When 100% of a rainfall event is retained by the PICP system, there is no pollution discharge to municipal storm drain systems or outfalls.

PICP systems are often a component of a treatment train where stormwater is managed and treated near its source and through conveyance infrastructure. When stormwater is infiltrated through a

PICP system, the surface first filters the water with additional assistance from water held in the bedding and aggregate layers.

In sufficient quantities, metals (e.g., copper and zinc) introduced into stormwater by human activities, such as weathering of roofs, rusting vehicles, or brake linings, can be toxic to aquatic life. Plant litter, animal waste, and fertilizers introduce nutrients (e.g., nitrogen and phosphorus) into stormwater, which can cause excessive and ecologically harmful plant and algae growth in receiving water systems. Suspended solids often contain heavy metals and nutrients; water quality benefits are provided when they are removed from the discharge. PICP systems do not typically remove dissolved forms of chemicals, salts, and metals (with some oxidation of nutrients), except by infiltration, as noted above.

Vehicular traffic introduces many other kinds of pollution into stormwater, including hydrocarbons. Stormwater that has passed through a PICP system generally contains fewer hydrocarbons than runoff from traditional pavements. In a PICP system, hydrocarbons (such as oils and grease) are removed from stormwater through a number of processes, including volatilization and biological degradation. Additionally, emulsified asphalts and similar products can provide a source of hydrocarbons in the discharge when compared to the products used in a PICP system.

Ecologically, increased runoff temperatures are another form of water pollution. During warmer months, urban environments introduce heat into stormwater. Heat from hot rooftops and pavements is transferred to stormwater during a runoff event and is conveyed to the receiving waters (i.e., creeks, rivers, and lakes). This process can create acute, rapid, and sometimes toxic increases in water temperature to downstream aquatic organisms. Water retained in the reservoir layers is often cooled down before its discharge. Water that infiltrates becomes part of the groundwater supply in the water cycle, further promoting temperature reduction.

With filtering as a primary means to reduce pollutants, total suspended solid reductions are often 75% to 90%, compared to reductions from conventional impervious pavements. Because nutrients, oils, and metals can be bound (adsorbed) to suspended particles, reductions of these pollutants are in part realized by trapping suspended solids in the jointing material, as well as sedimentation and decomposition within the pavement structure. The engineer may also consider pre- or posttreatment of runoff entering or exiting PICP through other means, such as bioswales or sand filters. This method can reduce total suspended solids entering the surface and assist in processing nitrogen and phosphorus (especially dissolved forms).

PICP pollutant reductions vary with the configuration of each site, pollutant sources, and contributing drainage area. There have been numerous full-scale monitoring studies of runoff and pollutant reduction characteristics in the United States, Canada, and overseas. Water volume and pollutant reductions are summarized in the ASCE book, *Permeable Pavements* (Eisenberg et al. 2015), as well as in other literature reviews. Monitoring and testing of stormwater may be desired by PICP owners to better evaluate the site-specific benefits to water quality (particularly for pilot or demonstration projects). For more information on stormwater quality monitoring, refer to the *Urban Stormwater BMP Performance Monitoring Manual* (Geosyntec 2009).

Increased impervious surfaces in an urban environment can cause hydromodification, an associated increase in the volume and duration of erosive flows downstream. By storing and infiltrating runoff, PICP can help address adverse effects of hydromodification. As a result, they can reduce downstream erosion and sedimentation, costs to protect downstream structures (e.g., bridges), and costs to remove sediment to maintain flood capacity.

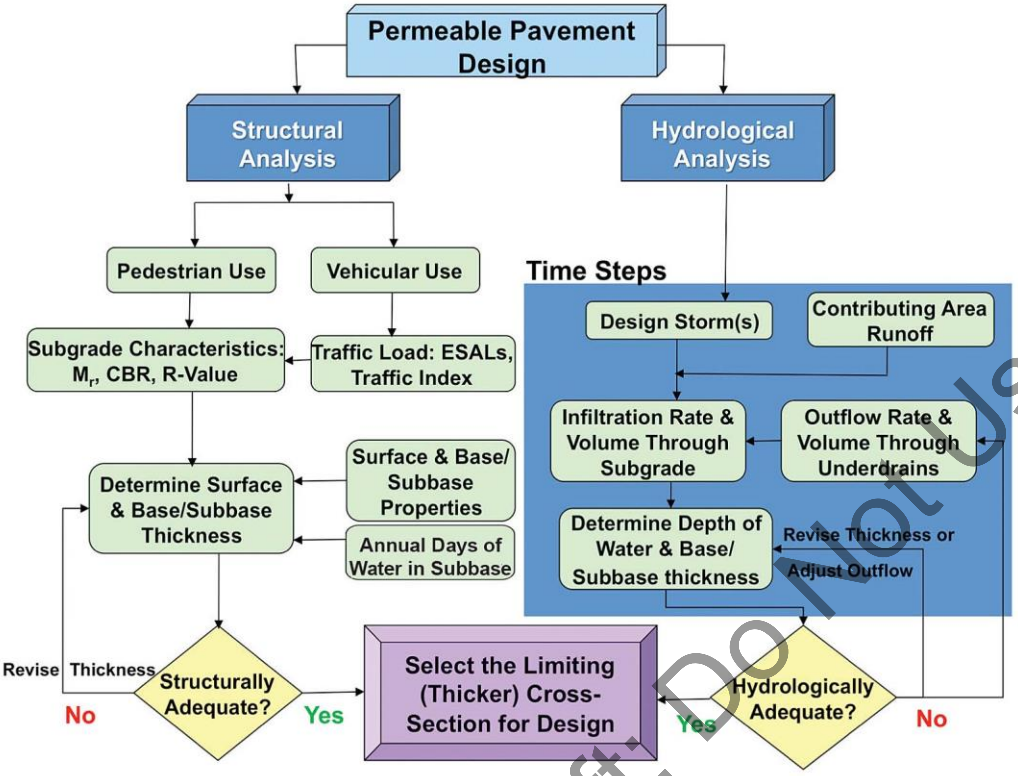


Figure 4-1. PICP Design Flowchart

Source: ICPI (2017b); reproduced with permission from Interlocking Concrete Pavement Institute.

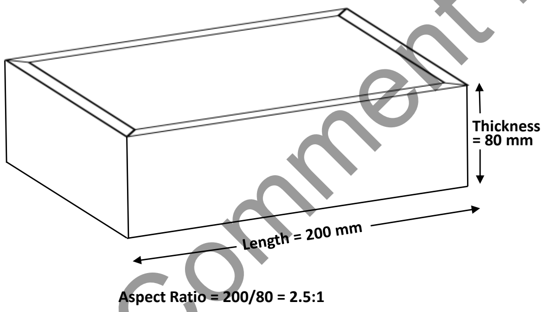
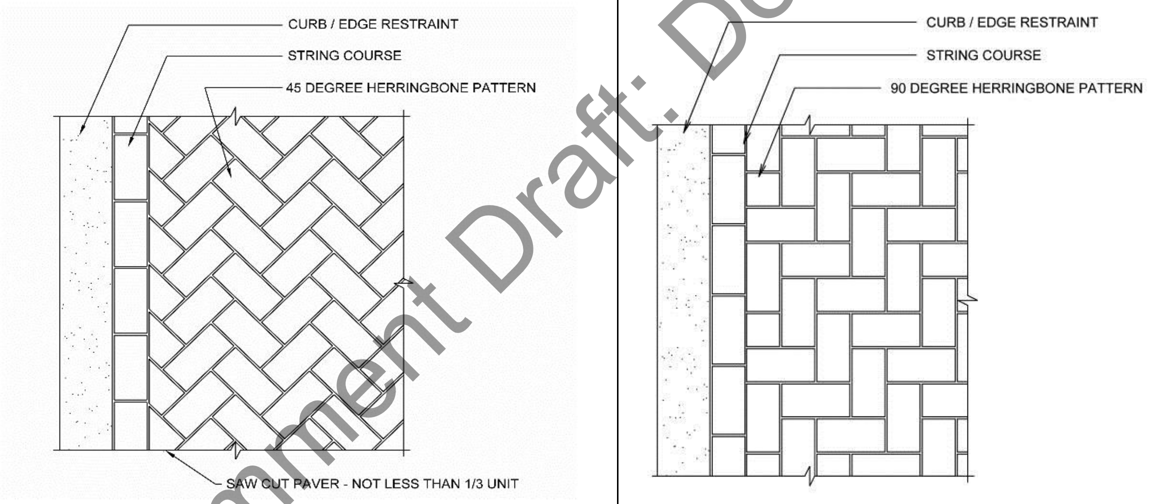


Figure 4-2. Aspect Ratio of a Paving Unit (Length/Thickness)

Source: ICPI (2017c); reproduced with permission from Interlocking Concrete Pavement Institute.

Figure 4-3. Example of Machine Installation of PICP

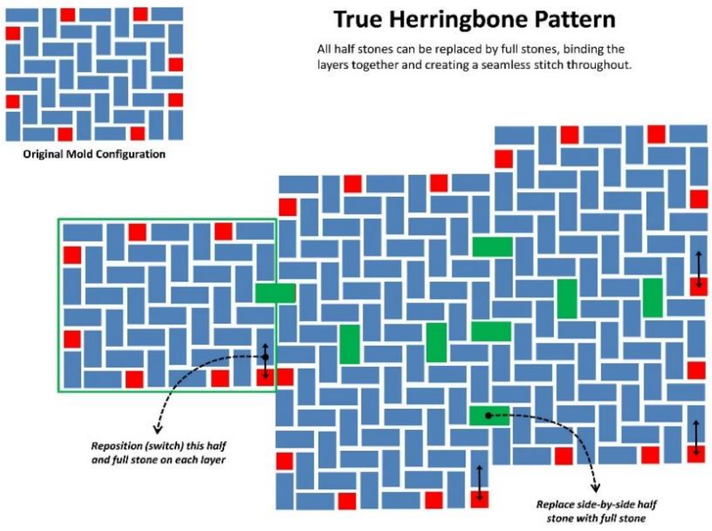
Source: ICPI (2012); reproduced with permission from Interlocking Concrete Pavement Institute.



(a) **(b)**

Figure 4-4. Two Herringbone Patterns with a Single Sailor Course set at (a) 45 degrees and (b) 90 degrees

Source: ICPI (2017b); reproduced with permission from Interlocking Concrete Pavement Institute.



True Herringbone Pattern

All half stones can be replaced by full stones, binding the layers together and creating a seamless stitch throughout.

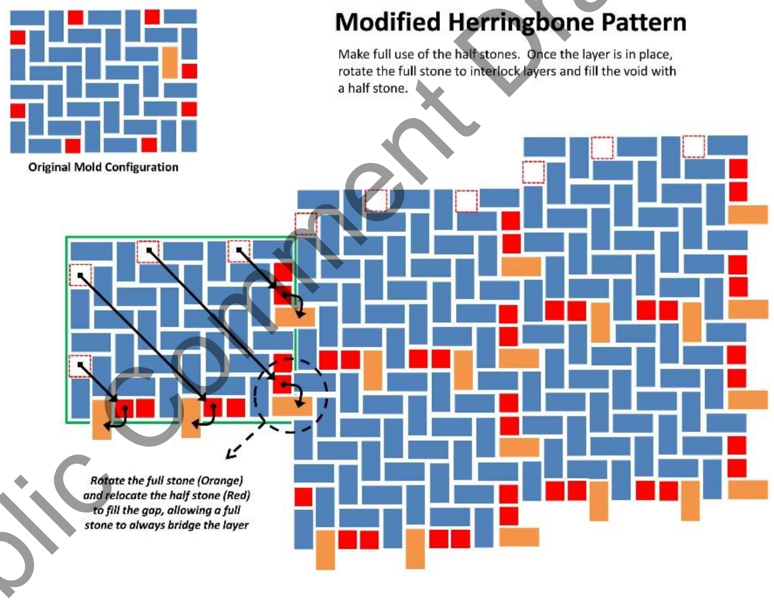
Original Mold Configuration

Reposition (switch) this half and full stone on each layer

Replace side-by-side half stone with full stone

Figure 4-5. Herringbone Pattern Layers Stitched with Whole Paving Units. Half-Sized Units Providing Stability During Shipping Are Removed Upon Installation of the Whole Paving Units

Source: ICPI (2017c); reproduced with permission from Interlocking Concrete Pavement Institute.



Modified Herringbone Pattern

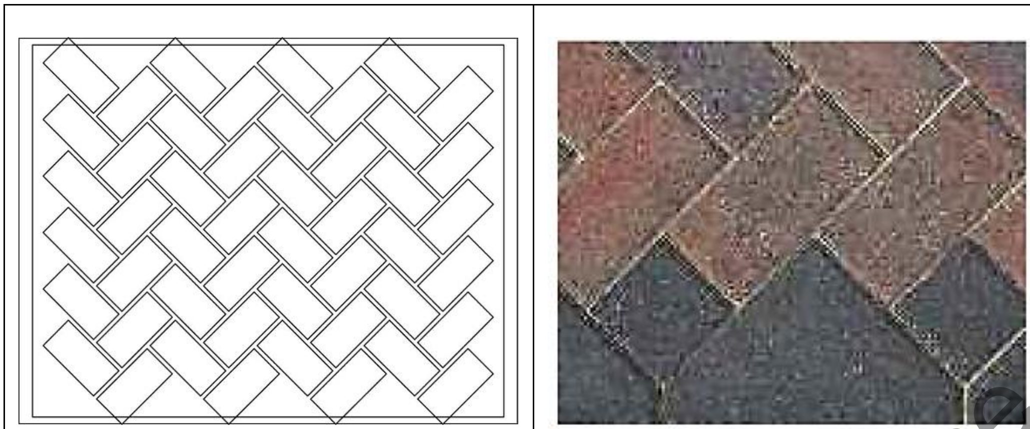
Make full use of the half stones. Once the layer is in place, rotate the full stone to interlock layers and fill the void with a half stone.

Original Mold Configuration

Rotate the full stone (Orange) and relocate the half stone (Red) to fill the gap, allowing a full stone to always bridge the layer

Figure 4-6. Stitching Herringbone Paver Layers with Whole Units. Half-Size Units Remain. No Joint Lines Are Longer than 1½ Paving Units

Source: ICPI (2017c); reproduced with permission from Interlocking Concrete Pavement Institute.



(a) (b)

Figure 4-7. (a) A 45-degree Herringbone Laying Pattern Does Not Require Stitching and (b) Manufactured Edge Pavers Called “Bishops Hats” Can Be Used or Rectangular Pavers Cut to Fit

Source: ICPI (2017c); reproduced with permission from Interlocking Concrete Pavement Institute.

Figure 4-8. A Stitching Pattern for L-Shaped Paving Units

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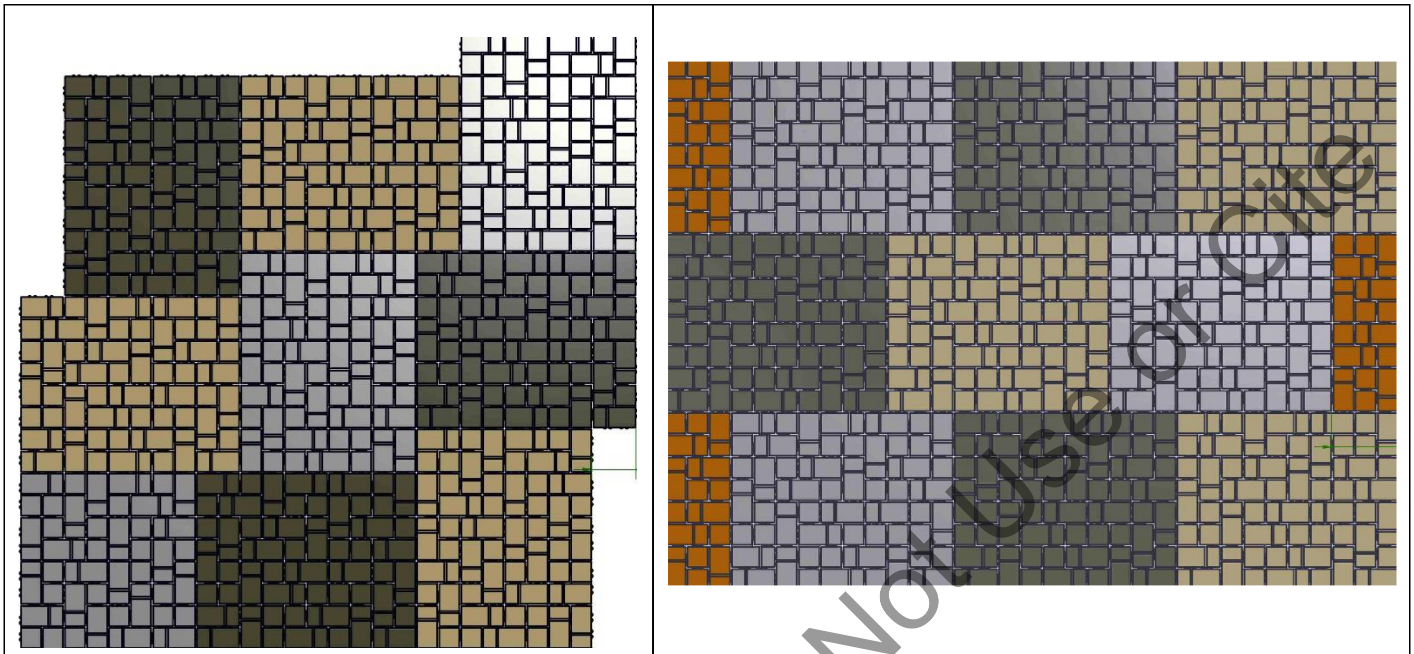

(a) (b)

Figure 4-9. Non-Stitched Laying Patterns to Increase Stability Under Automobile Traffic (a) Placed in a Herringbone Pattern and (b) Offset as a Running Bond Pattern

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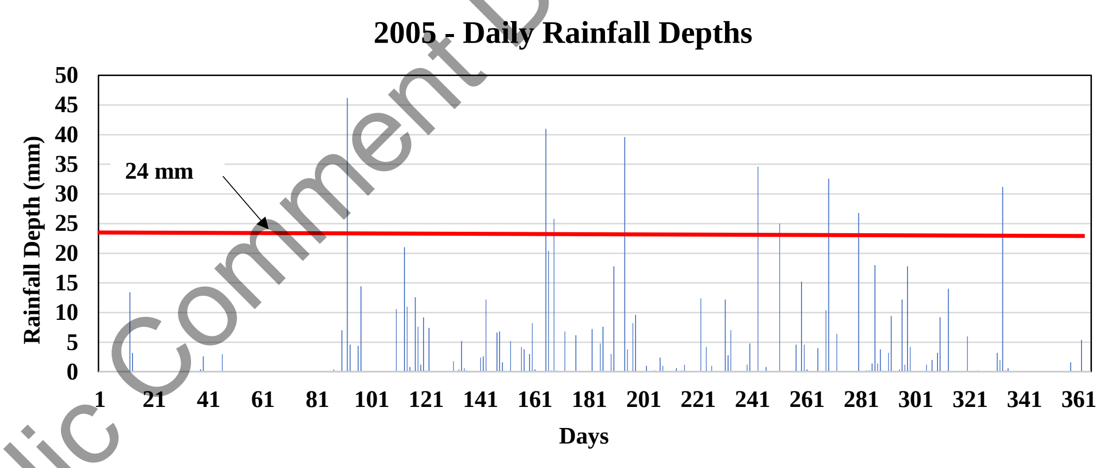

Figure 4-10. Finding the Annual Days Water Stands in the Subbase

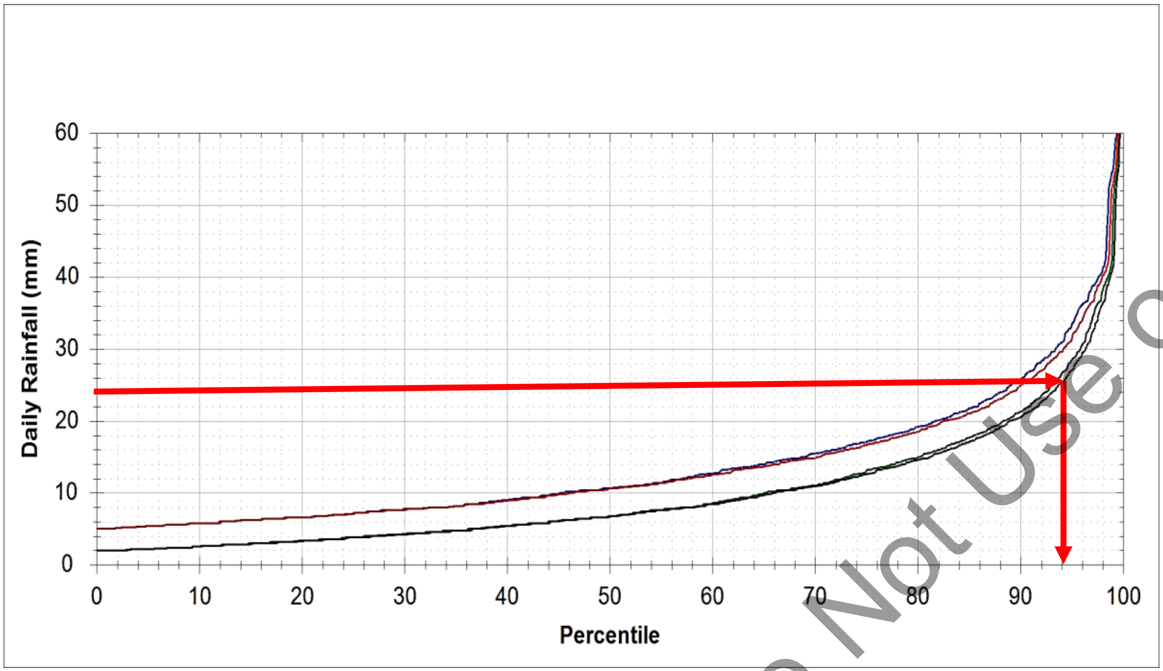

Figure 4-11. Using a Rainfall Frequency Spectrum Graph to Determine the Average Annual Days Water Stands in the Subbase

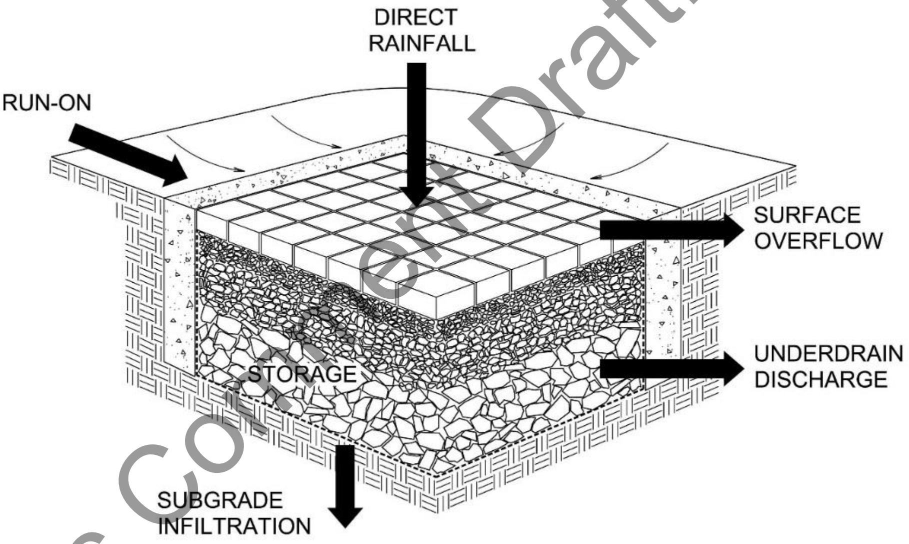

Figure 4-12. Water Balance Diagram

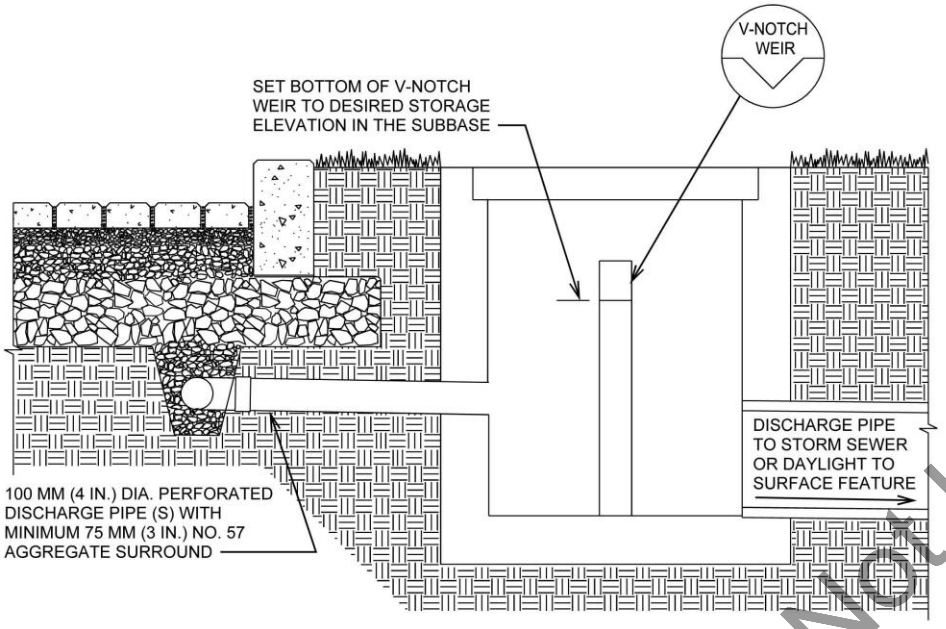

Figure 4-13. Example Outlet Structure to Control the Depth of Water in the PICP

Source: ICPI (2017e); reproduced with permission from Interlocking Concrete Pavement Institute.

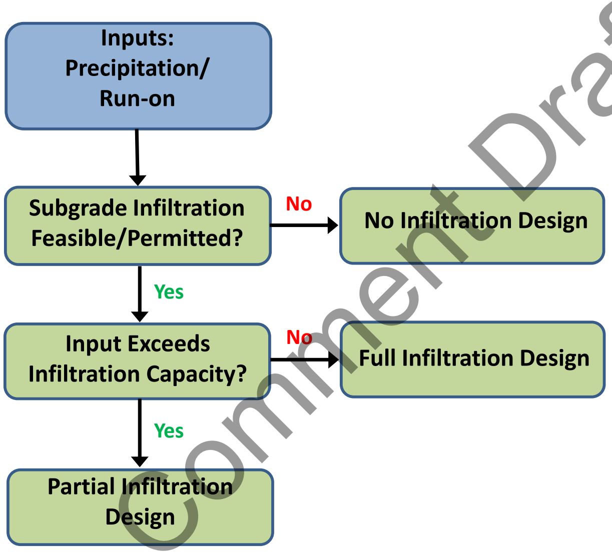

Figure 4-14. Selection of PICP System Type

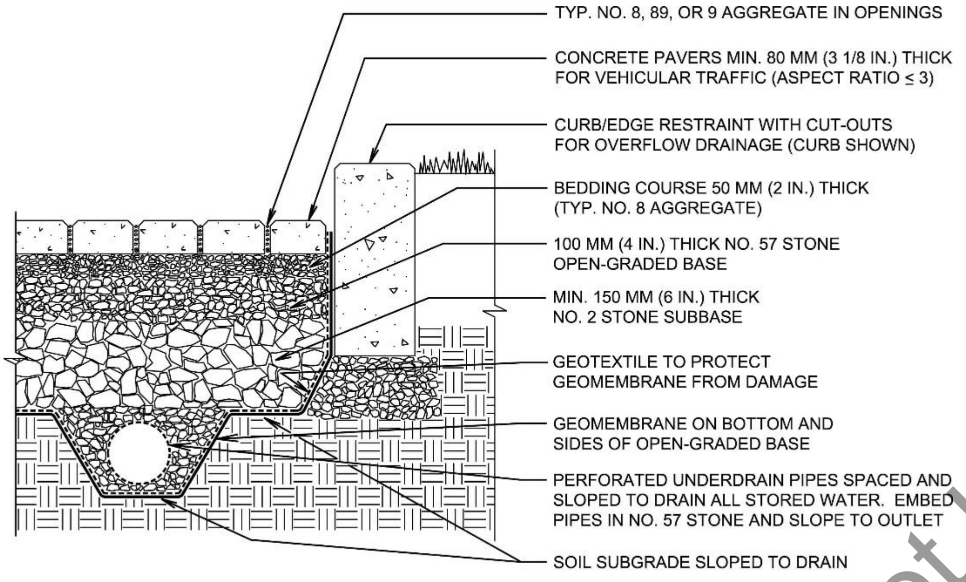

Figure 4-15. No-Infiltration System Design

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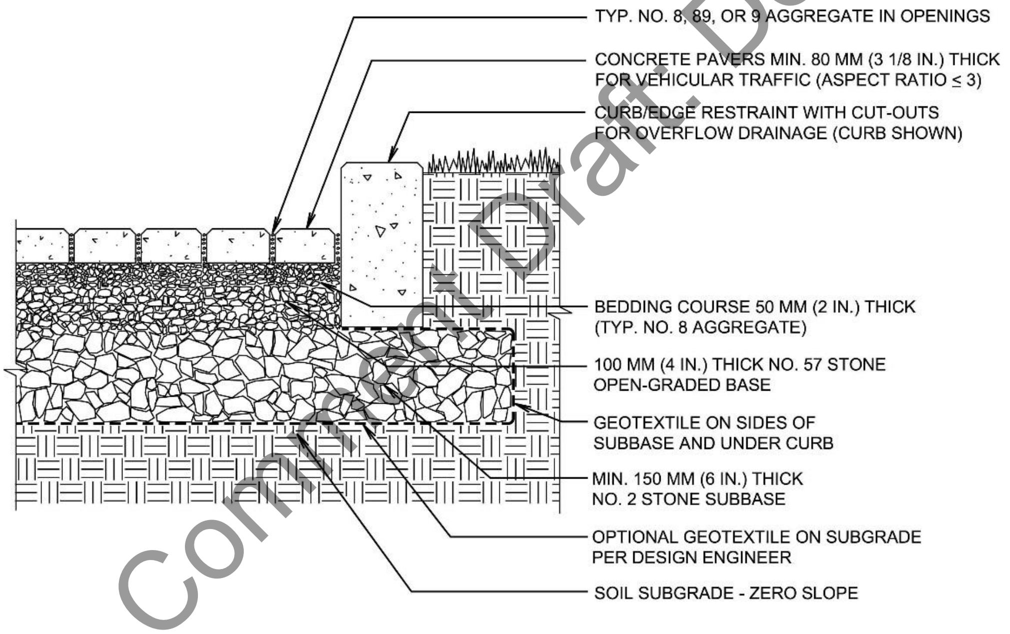

Figure 4-16. Full-Infiltration System Design (with no Underdrains)

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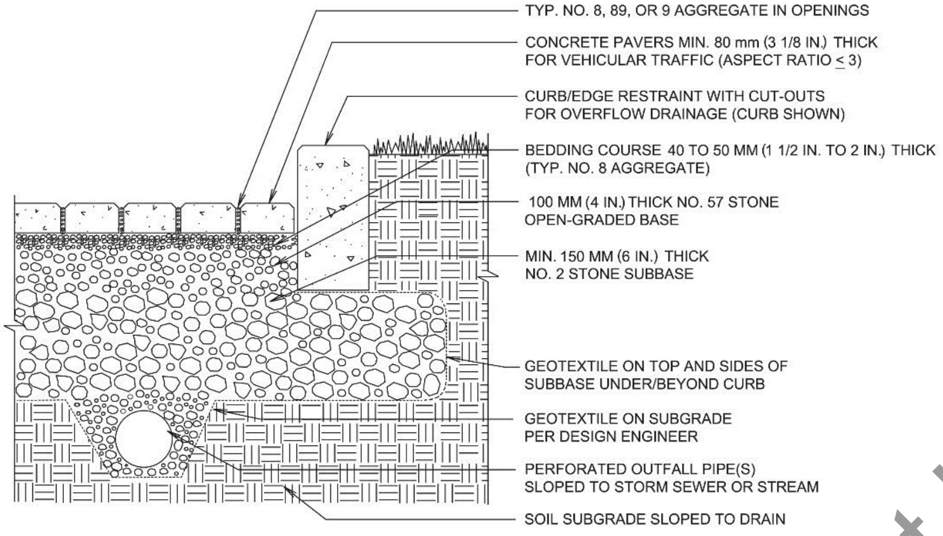

Figure 4-17. Partial-Infiltration System Design

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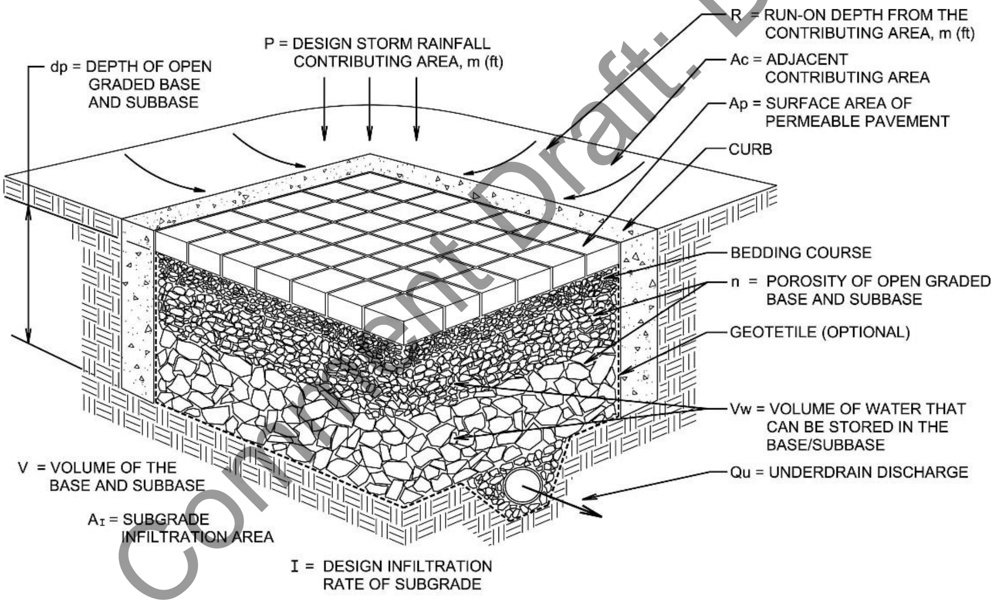

Figure 4-18. Symbols Used in Sizing the Subbase for Water Storage

A_c = Surface area of the adjacent contributing area, m^2 (sf). Note: where acres are reported, multiply the value by 43,560 to convert to square feet.

A_p = Surface area of the permeable interlocking concrete pavement, m^2 (sf)

A_I = Area of subgrade infiltration, m^2 (sf). Commonly $A_P \approx A_I$. In situations where a portion of the subgrade is either lined (e.g., a PICP area adjacent to a building) or is not conducive to infiltration (e.g., highly sloped), the reduced subgrade infiltration area is adjusted. Alternately, where the subbase extends beyond the curb line the increased subgrade infiltration area can be used.

C = Runoff Coefficient for the contributing area, which represents the ratio of runoff to rainfall. The value of C is between 0 and 1. Reference the Rational Method for the runoff coefficient(s) based on land cover or use agency values.

d_p = Depth of open graded stone base/subbase reservoir, m (ft). Note that the depth does not include the bedding course or pavers. The minimum allowable value of d_p must also satisfy the base/subbase thickness determined in the structural analysis.

I = Design infiltration rate in m/h (ft/h) of the subgrade soil under the pavement. Design infiltration rates are based on in-situ infiltration tests and adjusted by an infiltration reduction factor per engineering judgment. Note: where in./h is reported, divide the value by 12 to convert to ft/h.

n = Porosity of the base and subbase aggregates (typically 0.4).

P = Design storm rainfall depth, m (ft)

Q_U = Outflow rate through the underdrain(s), m^3/h (cf/h). Examples of orifice and common weir equations are included in Appendix C. Note: where cubic feet per second is reported, multiply the value by 3,600 to convert to cubic feet per hour.

R = Run-on depth from the contributing area, m (ft) which is the design storm rainfall depth (P) multiplied by the runoff coefficient (C) of the contributing area. $R \leq P$.

T_D = the maximum post storm drawdown time for the base/subbase (h). The recommended maximum time for the base/subbase drawdown is typically defined by the local regulatory agency, generally no more than 72 h including the rainstorm.

T_s = Duration of the design storm (h). Typically, a 24-h design storm is used for sizing, but it can range from a 2-h to a 24-h event.

V = Volume of the base/subbase, m^3 (cf)

V_w = Volume of water that can be stored in the base/subbase, m^3 (cf)

Z = Outlet elevation factor (dimensionless). When the underdrain is raised above the subgrade, Z represents the percentage of time when underdrain flow exists.

Source: ICPI (2017b); reproduced with permission from Interlocking Concrete Pavement Institute.

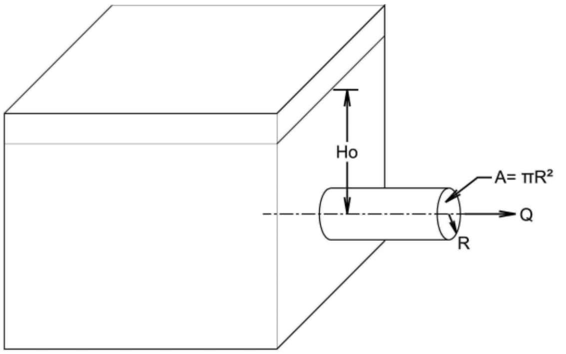

Figure 4-19. Symbols Used in the Orifice Equation

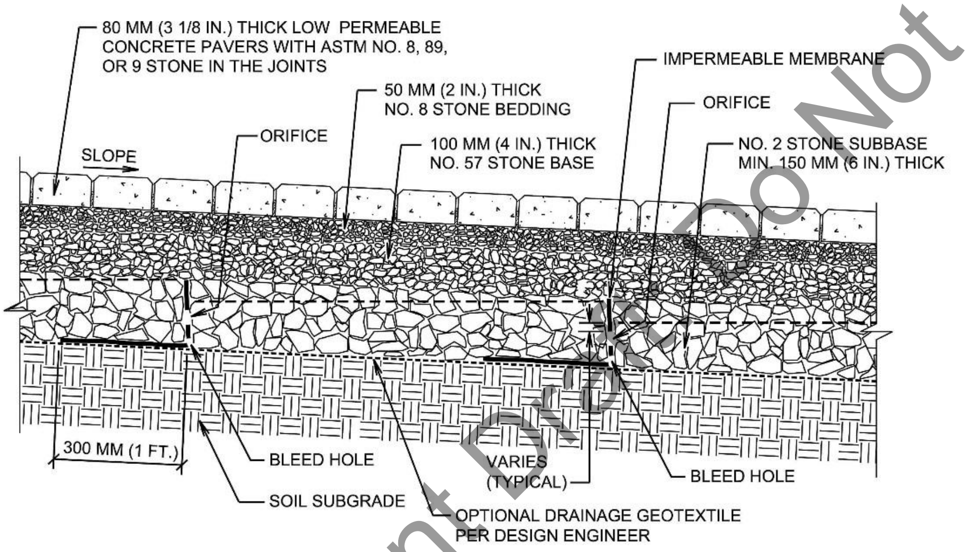

Figure 4-20. Example of Water Flow Barriers / Check Dams with Geomembrane

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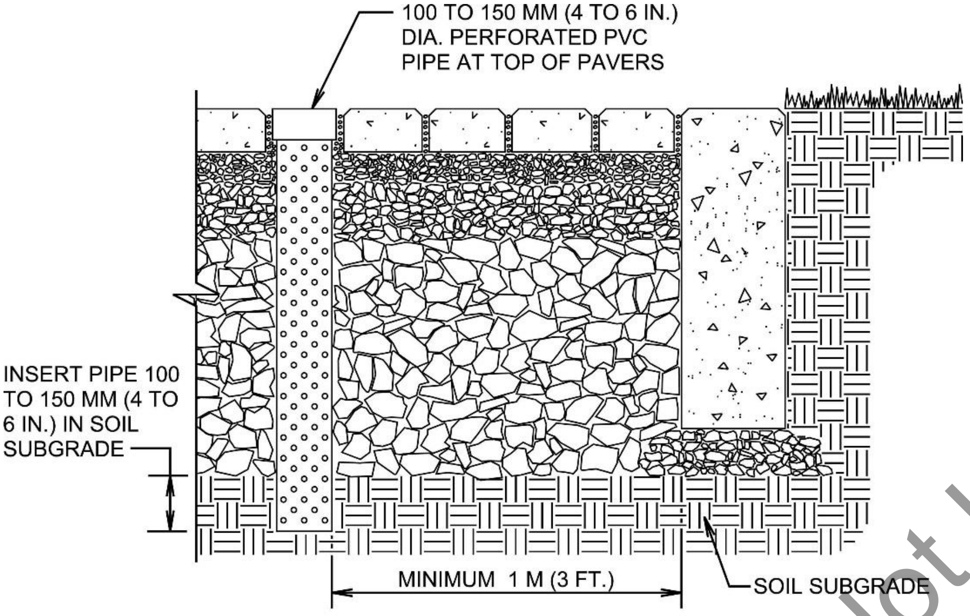

Figure 4-21. Observation Well in PICP at the Lowest Subgrade Elevation

Source: ICPI (2017b); reproduced with permission from Interlocking Concrete Pavement Institute.

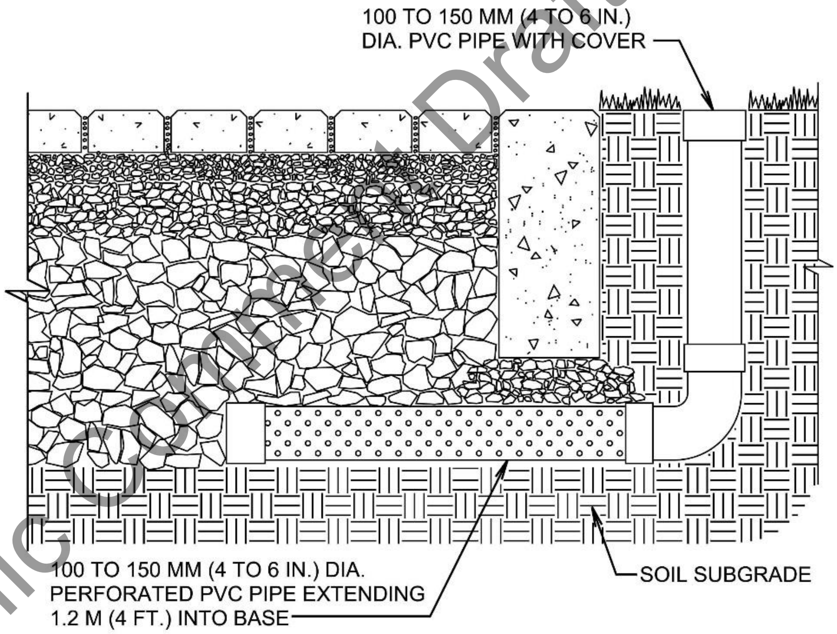

Figure 4-22. Observation Well Extending Outside PICP at the Lowest Subgrade Elevation

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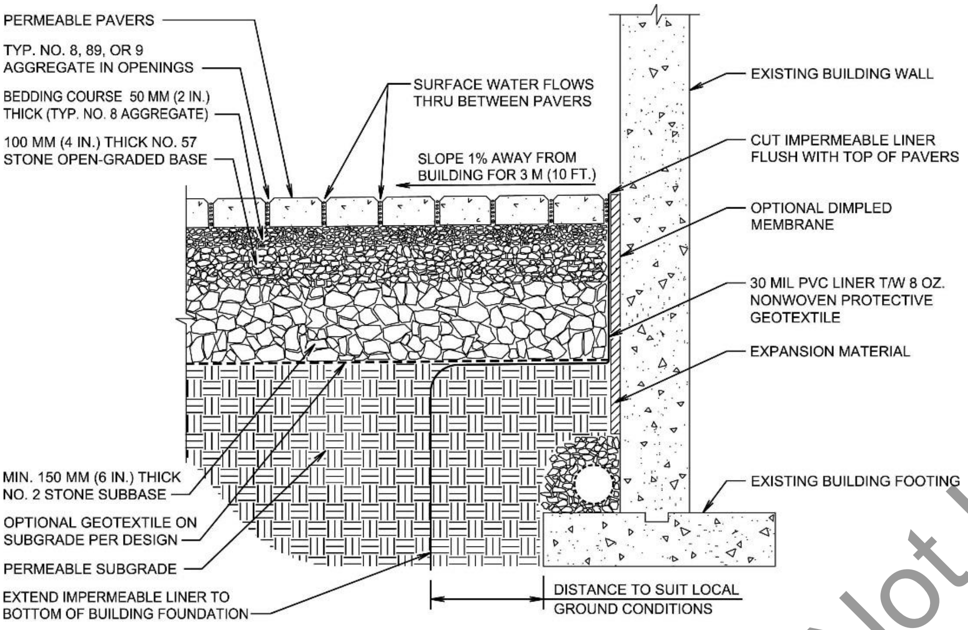

Figure 4-23. Example Foundation Wall Protection from Water Infiltration

Source: ICPI (2017b); reproduced with permission from Interlocking Concrete Pavement Institute.

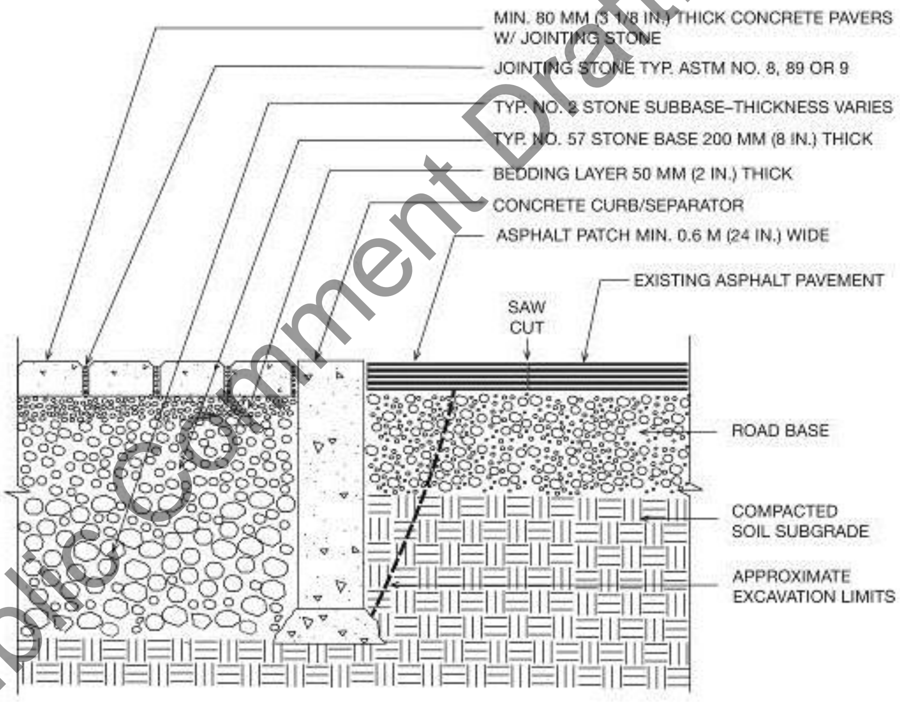

Figure 4-24. Flexible Pavement Transition Detail

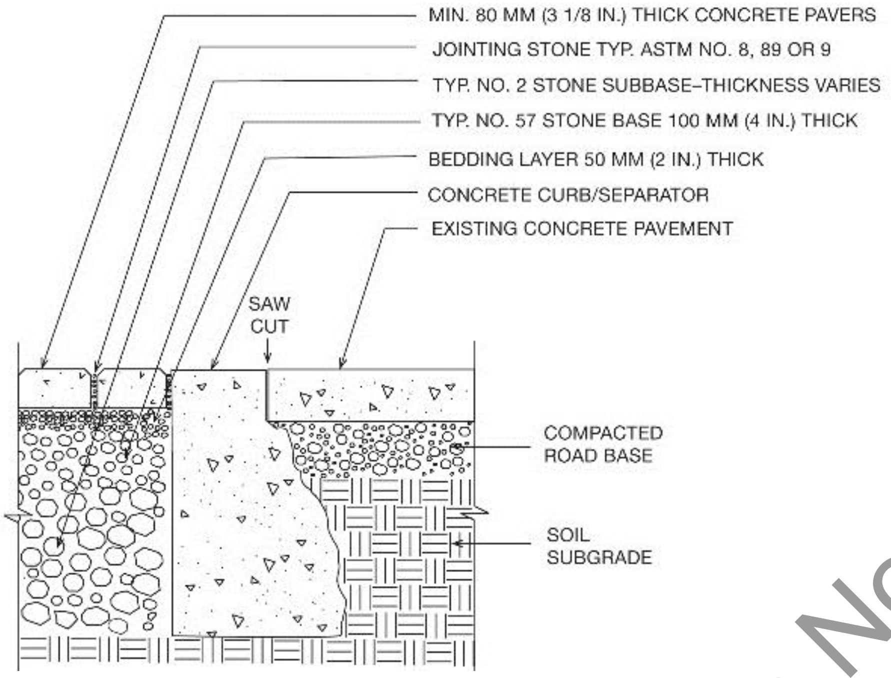

Figure 4-25. Rigid Pavement Transition Detail

Table 4-1. Roadway and Pavement Classifications and Typical Design Traffic.

Pavement Class	*Description*	*Design ESALs*	*Design TI*
Arterial	Through traffic with access to high-density, regional, commercial, and office developments or downtown streets. General traffic mix.	9,000,000	11.5
Major collector	Traffic with access to low-density, local, commercial, and office development or high-density, residential subdivisions. General traffic mix.	3,000,000	10
Minor collector	Through traffic with access to low-density, neighborhood, commercial development or low-density, residential subdivisions. General traffic mix.	1,000,000	9
Bus passenger drop-off	Public transport centralized facility for buses to pick up passengers from other modes of transport, or for parking of city or school buses.	500,000	8.5
Local commercial	Commercial and limited through traffic with access to commercial premises and multi-family and single-family residential roads. Used by private automobiles, service vehicles and heavy delivery trucks This category includes large parking lots at commercial retail facilities.	330,000	8
Residential	No through traffic with access to multi-family and single-family residential properties. Used by private automobiles, service vehicles and light delivery trucks, including limited construction traffic.	110,000	7
Facility parking and alleys	Parking areas for private automobiles at large facilities with access for emergency vehicles and occasional use by service vehicles or heavy delivery trucks.	90,000	7
Commercial parking	Restricted parking and drop-off areas associated with business premises, mostly used by private automobiles and occasional light delivery trucks. No construction traffic over finished surfaces.	30,000	6
Commercial plaza	Predominantly pedestrian traffic, but with access for occasional heavy maintenance and emergency vehicles. No construction traffic over finished surfaces.	10,000	5

Source: Courtesy of Brick Industry Association (2003); reproduced with permission.

Table 4-2. ASTM Standard Sizes of Aggregate (Metric)

Size Number	*Nominal Size (mm)*	*100 mm*	*90 mm*	*75 mm*	*63 mm*	*50 mm*	*37.5 mm*	*25.0 mm*	*19.0 mm*	*12.5 mm*	*9.5 mm*	*4.75 mm*	*2.36 mm*	*1.18 mm*	*300 µm*	*150 µm*
1	90 to 37.5	100	90–100		25–60		0–15		0–5							
2	63 to 37.5			100	90–100	35–70	0–15		0–5							
24	63 to 19.0			100	90–100		25–60		0–10	0–5						
3	50 to 25.0				100	90–100	35–70	0–15		0–5						
357	50 to 4.75				100	95–100		35–70		10–30		0–5				
4	37.5 to 19.0					100	90–100	20–55	0–15		0–5					
467	37.5 to 4.75					100	95–100		35–70		10–30	0–5				
5	25.0 to 12.5						100	90–100	20–55	0–10	0–5					
56	25.0 to 9.5						100	90–100	40–85	10–40	0–15	0–5				
57	25.0 to 4.75						100	95–100		25–60		0–10	0–5			
6	19.0 to 9.5							100	90–100	20–55	0–15	0–5				
67	19.0 to 4.75							100	90–100		20–55	0–10	0–5			
68	19.0 to 2.36							100	90–100		30–65	5–25	0–10	0–5		
7	12.5 to 4.75								100	90–100	40–70	0–15	0–5			
78	12.5 to 2.36								100	90–100	40–75	5–25	0–10	0–5		
8	9.5 to 2.36									100	85–100	10–30	0–10	0–5		
89	9.5 to 1.18									100	90–100	20–55	5–30	0–10	0–5	
9	4.75 to 1.18										100	85–100	10–40	0–10	0–5	
10	4.75 to 0.075										100	85–100				10–30

Source: ASTM (2012a); reproduced with permission from ASTM.

Table 4-3. ASTM Standard Sizes of Aggregate (U.S. Customary)

Size Number	*Nominal Size (inches)*	*4 in.*	*3.5 in.*	*3 in.*	*2.5 in.*	*2 in.*	*1.5 in.*	*1 in.*	*3/4 in.*	*1/2 in.*	*3/8 in.*	*No. 4*	*No. 8*	*No. 16*	*No. 50*	*No. 100*
1	3.5 to 1.5	100	90–100		25–60		0–15		0–5							
2	2.5 to 1.5			100	90–100	35–70	0–15		0–5							
24	2.5 to 3/4			100	90–100		25–60		0–10	0–5						
3	2 to 1				100	90–100	35–70	0–15		0–5						
357	2 to No. 4				100	95–100		35–70		10–30		0–5				
4	1.5 to 3/4					100	90–100	20–55	0–15		0–5					
467	1.5 to No. 4					100	95–100		35–70		10–30	0–5				
5	1 to 1/2						100	90–100	20–55	0–10	0–5					
56	1 to 3/8						100	90–100	40–85	10–40	0–15	0–5				
57	1 to No. 4						100	95–100		25–60		0–10	0–5			
6	3/4 to 3/8							100	90–100	20–55	0–15	0–5				
67	3/4 to No. 4							100	90–100		20–55	0–10	0–5			
68	3/4 to No. 8							100	90–100		30–65	5–25	0–10	0–5		
7	1/2 to No. 4								100	90–100	40–70	0–15	0–5			
78	1/2.to No. 8								100	90–100	40–75	5–25	0–10	0–5		
8	3/8 to No. 8									100	85–100	10–30	0–10	0–5		
89	3/8 to No. 16									100	90–100	20–55	5–30	0–10	0–5	
9	No. 4 to No. 16										100	85–100	10–40	0–10	0–5	
10	No. 4 to No. 200										100	85–100				10–30

Source: ASTM (2012a); reproduced with permission from ASTM.

Table 4-4. CSA Grading Requirements for Coarse Aggregate (Metric)

	Total Passing Each Sieve, Percentage by Mass										
Nominal Size	*112 mm*	*80 mm*	*56 mm*	*40 mm*	*28 mm*	*20 mm*	*14 mm*	*10 mm*	*5 mm*	*2.5 mm*	*1.25 mm*
Group I											
40-5			100	95–100		35–70		10–30	0–5		
28-5				100	95–100		30–65		0–10	0–5	
20-5					100	85–100	60–90	25–60	0–10	0–5	
14-5						100	90–100	45–75	0–15	0–5	
10-2.5							100	85–100	10–30	0–10	0–5
Group II											
80-40	100	90–100	25–60	0–15		0–5					
56-28		100	90–100	30–65	0–15		0–5				
40-20			100	90–100	25–60	0–15		0–5			
28-14				100	90–100	30–65	0–15		0–5		
20-10					100	85–100		0–20	0–5		
14-10						100	85–100	0–45	0–10		
10-5							100	85–100	0–20	0–5	
5-2.5								100	70–100	10–40	0–10

Note: Group I comprises combined aggregate gradings most commonly used in concrete production. Group II provides for special requirements (i.e., gap grading or pumping, or for blending two or more sizes to produce Group I gradings).

Source: CSA (2014a); reproduced with permission from CSA Group, © Canadian Standards Association.

Table 4-5. CSA Grading Requirements for Coarse Aggregate (U.S. Customary)

	Total passing each Sieve, percentage by mass										
Nominal Size	*4.4 in.*	*3.15 in.*	*2.2 in.*	*1.5 in.*	*1.1 in.*	*0.8 in.*	*0.55 in.*	*0.4 in.*	*0.2 in.*	*0.1 in.*	*0.05 in.*
Group I											
1.6-0.2			100	95–100		35–70		10–30	0–5		
1.1-0.2				100	95–100		30–65		0–10	0–5	
0.8-0.2					100	85–100	60–90	25–60	0–10	0–5	
0.55-0.2						100	90–100	45–75	0–15	0–5	
0.4-0.1							100	85–100	10–30	0–10	0–5
Group II											
3.15-1.5	100	90–100	25–60	0–15		0–5					
2.2-1.1		100	90–100	30–65	0–15		0–5				
1.5-0.8			100	90–100	25–60	0–15		0–5			
1.1-0.55				100	90–100	30–65	0–15		0–5		
0.8-0.4					100	85–100		0–20	0–5		
0.55-0.4						100	85–100	0–45	0–10		
0.4-0.2							100	85–100	0–20	0–5	
0.2-0.1								100	70–100	10–40	0–10

Note: Group I comprises combined aggregate gradings most commonly used in concrete production. Group II provides for special requirements (i.e., gap grading or pumping, or for blending two or more sizes to produce Group I gradings).

Source: CSA (2014a); reproduced with permission from CSA Group, © Canadian Standards Association.

Table 4-6. Summary of Rut Models Developed for Different Layers in a PICP

Layer	*Rut Model*	*Moisture Condition*	*Model Parameters* *a*	*b*	*c*
Combined bedding and base	$RD_{BB} = a \times h\_SB + b$	Dry	0	4.0	—
		Wet	–0.012	13.1	—
Subbase	$RD_{SB} = (a \times SSR^{b}) \times N^{c}$	Dry	3.10×10^{-6}	2.70	1
		Wet	3.10×10^{-6}	2.70	1
Subgrade (silty clay)	$RD_{SG} = (a \times SSR + b) \times N^{c}$	Dry	0.03	–0.01	0.5
		Wet	0.03	–0.01	0.5

Notes: RD_{xx}, rut depth of *xx* layer (*BB*=surface (paver, bedding, and base); *SB*=subbase; *SG* = subgrade), mm; *h_SB*, thickness of subbase, mm; *SSR*, shear stress–strength ratio at the top of the layer; *N*, load repetition; and *a*, *b*, *c*, model constants.

Source: UCPRC (2014); reproduced with permission from University of California Pavement Research Center.

Table 4-7. Summary of Inputs for Performance Modeling and M-E Design of PICP

Variable		*Surface (Paver, Bedding, and Base)*		*Subbase*			*Subgrade*	
		Thickness (mm)	*Stiffness (MPa[a])*	*Thickness (mm)*	*Stiffness (MPa)*	c, *ϕ* (*kPa, °*)	*Stiffness (MPa)*	c, *ϕ* (*kPa, °*)
Pavement Structure and Materials	Label	h1	E1	h2	E2	*c*, ϕ	E3	*c*, ϕ
	Value	230	110 (dry) 87 (wet)	Varying (450 default)	122 (dry) 73 (wet)	0, 45 (dry) 0, 30 (wet)	60 (dry) 37 (wet)	15, 25 (dry) 9, 15 (wet)
Climate	*Variable*	*Wet Days[b]*						
	Label	W						
	Value	50						
Traffic	*Variable*	*Axle Type*	*Axle Load[c] (kN)[d]*	*Stress Location*				
	Label	AT	AL	SL				
	Value	Single (S) Tandem (T)	10 to 160 (S) 20 to 200 (T)	Under Wheel Between Wheel				

[a] 6.890 MPa = 1,000 psi.

[b] Number of days in a calendar year when the subbase has standing water.

[c] The total truck traffic volume was divided into different axle loads according to an axle–load distribution factor. Group 1 WIM truck traffic data from California were used as the default axle–load distribution factor.

[d] 4.448 kN = 1,000 lb.

Source: UCPRC (2014); reproduced with permission from University of California Pavement Research Center.

Table 4-8. Subbase Thickness Design Table (Metric)

		Number of Days in a Year That Water Stands in Subbase															
		0				*≤10*				*11–30*				*31–50*			
Subgrade Resilient Modulus, MPa (CBR)	Dry	40	60	80	100	40	60	80	100	40	60	80	100	40	60	80	100
	Wet	24 (1.6)	36 (3)	48 (4.8)	60 (6.8)	24 (1.6)	36 (3)	48 (4.8)	60 (6.8)	24 (1.6)	36 (3)	48 (4.8)	60 (6.8)	24 (1.6)	36 (3)	48 (4.8)	60 (6.8)
Lifetime ESALs (Traffic Index)		Minimum subbase thickness in mm ASTM No. 2 for 25-mm allowable rut depth (All subbases are under 100-mm-thick ASTM No. 57 base, under 50-mm-thick ASTM No. 8 bedding layer under 80-mm-thick concrete pavers.)															
50,000 (6.3)		150	150	150	150	150	150	150	150	150	150	150	150	175	150	150	150
100,000 (6.8)		150	150	150	150	210	150	150	150	260	150	150	150	285	180	150	150
200,000 (7.4)		230	150	150	150	315	210	150	150	365	255	160	150	395	285	185	150
300,000 (7.8)		290	180	150	150	375	265	170	150	425	315	215	150	455	340	240	160
400,000 (8.1)		330	220	150	150	420	305	210	150	470	350	255	175	500	380	280	200
500,000 (8.3)		360	250	160	150	450	335	240	160	500	380	280	205	530	410	305	230
600,000 (8.5)		385	275	185	150	475	360	260	180	525	405	305	225	555	435	330	250
700,000 (8.6)		410	295	205	150	495	380	280	200	550	425	325	245	580	455	350	270
800,000 (8.8)		425	310	220	150	515	395	295	215	565	440	340	260	600	470	365	285
900,000 (8.9)		440	325	235	155	530	410	310	230	585	455	355	270	615	485	380	295
1,000,000 (9.0)		455	340	250	165	545	425	325	240	600	470	365	285	630	500	390	310

Source: UCPRC (2014); reproduced with permission from University of California Pavement Research Center.

Table 4-8 (*Continued*). Subbase Thickness Design Table (Metric)

		Number of Days in a Year That Water Stands in Subbase															
		51–70				*71–90*				*91–110*				*111–130*			
Subgrade Resilient Modulus, MPa (CBR)	Dry	40	60	80	100	40	60	80	100	40	60	80	100	40	60	80	100
	Wet	24 (1.6)	36 (3)	48 (4.8)	60 (6.8)	24 (1.6)	36 (3)	48 (4.8)	60 (6.8)	24 (1.6)	36 (3)	48 (4.8)	60 (6.8)	24 (1.6)	36 (3)	48 (4.8)	60 (6.8)
Lifetime ESALs (Traffic Index)		Minimum subbase thickness in mm ASTM No. 2 for 25-mm allowable rut depth (All subbases are under 100-mm-thick ASTM No. 57 base, under 50-mm-thick ASTM No. 8 bedding layer under 80-mm-thick concrete pavers.)															
50,000 (6.3)		195	150	150	150	210	150	150	150	225	150	150	150	235	150	150	150
100,000 (6.8)		310	200	150	150	325	215	150	150	335	230	150	150	350	240	150	150
200,000 (7.4)		415	305	205	150	430	320	215	150	445	330	230	150	455	340	240	160
300,000 (7.8)		475	360	260	180	495	375	275	195	505	390	285	210	520	400	295	220
400,000 (8.1)		520	400	295	220	535	415	310	235	550	430	325	245	565	440	335	255
500,000 (8.3)		550	430	325	245	570	445	340	260	585	460	350	270	595	470	360	280
600,000 (8.5)		580	455	350	270	595	470	360	280	610	485	375	295	625	495	385	305
700,000 (8.6)		600	475	365	285	620	490	380	300	635	505	395	310	645	515	405	320
800,000 (8.8)		620	490	385	300	640	505	395	315	655	520	410	330	665	535	420	340
900,000 (8.9)		635	505	395	315	655	525	410	330	670	535	425	340	685	550	435	350
1,000,000 (9.0)		650	520	410	325	670	535	425	340	685	550	435	355	700	560	445	365

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Table 4-9. Subbase Thickness Design Table (U.S. Customary Units)

		Number of Days in a Year That Water Stands in Subbase															
		0				*≤10*				*11–30*				*31–50*			
Subgrade Resilient Modulus, ksi (CBR)	Dry	5.8	8.7	11.6	14.5	5.8	8.7	11.6	14.5	5.8	8.7	11.6	14.5	5.8	8.7	11.6	14.5
	Wet	3.5 (1.6)	5.2 (3)	6.7 (4.8)	8.7 (6.8)	3.5 (1.6)	5.2 (3)	6.7 (4.8)	8.7 (6.8)	3.5 (1.6)	5.2 (3)	6.7 (4.8)	8.7 (6.8)	3.5 (1.6)	5.2 (3)	6.7 (4.8)	8.7 (6.8)
Lifetime ESALs (Traffic Index)		Minimum subbase thickness in mm ASTM No. 2 for 25-mm allowable rut depth (All subbases are under 100-mm-thick ASTM No. 57 base, under 50-mm-thick ASTM No. 8 bedding layer under 80-mm-thick concrete pavers.)															
50,000 (6.3)		6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	7.0	6.0	6.0	6.0
100,000 (6.8)		6.0	6.0	6.0	6.0	8.5	6.0	6.0	6.0	10.5	6.0	6.0	6.0	11.5	7.0	6.0	6.0
200,000 (7.4)		9.0	6.0	6.0	6.0	12.5	8.5	6.0	6.0	14.5	10.0	6.5	6.0	16.0	11.5	7.5	6.0
300,000 (7.8)		11.5	7.0	6.0	6.0	15.0	10.5	7.0	6.0	17.0	12.5	8.5	6.0	18.0	13.5	9.5	6.5
400,000 (8.1)		13.0	9.0	6.0	6.0	17.0	12.0	8.5	6.0	19.0	14.0	10.0	7.0	20.0	15.0	11.0	8.0
500,000 (8.3)		14.5	10.0	6.5	6.0	18.0	13.5	9.5	6.5	20.0	15.0	11.0	8.0	21.0	16.5	12.0	9.0
600,000 (8.5)		15.5	11.0	7.5	6.0	19.0	14.5	10.5	7.0	21.0	16.0	12.0	9.0	22.0	17.5	13.0	10.0
700,000 (8.6)		16.5	12.0	8.0	6.0	20.0	15.0	11.0	8.0	22.0	17.0	13.0	10.0	23.0	18.0	14.0	11.0
800,000 (8.8)		17.0	12.5	9.0	6.0	20.5	16.0	12.0	8.5	22.5	17.5	13.5	10.5	24.0	19.0	14.5	11.5
900,000 (8.9)		17.5	13.0	9.5	6.0	21.0	16.5	12.5	9.0	23.5	18.0	14.0	11.0	24.5	19.5	15.0	12.0
1,000,000 (9.0)		18.0	13.5	10.0	6.5	22.0	17.0	13.0	9.5	24.0	19.0	14.5	11.5	25.0	20.0	15.5	12.5

Note: Subbase thickness is calculated by dividing metric thicknesses in Table 4-9 by 25 and rounding to nearest 0.5 in.

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Table 4-9 (*Continued*). Subbase Thickness Design Table (U.S. Customary Units)

		Number of Days in a Year That Water Stands in Subbase															
		51–70				*71–90*				*91–110*				*111–130*			
Subgrade Resilient Modulus, ksi (CBR)	Dry	5.8	8.7	11.6	14.5	5.8	8.7	11.6	14.5	5.8	8.7	11.6	14.5	5.8	8.7	11.6	14.5
	Wet	3.5 (1.6)	5.2 (3)	6.7 (4.8)	8.7 (6.8)	3.5 (1.6)	5.2 (3)	6.7 (4.8)	8.7 (6.8)	3.5 (1.6)	5.2 (3)	6.7 (4.8)	8.7 (6.8)	3.5 (1.6)	5.2 (3)	6.7 (4.8)	8.7 (6.8)
Lifetime ESALs (Traffic Index)		Minimum subbase thickness in mm ASTM No. 2 for 25-mm allowable rut depth (All subbases are under 100-mm-thick ASTM No. 57 base, under 50-mm-thick ASTM No. 8 bedding layer under 80-mm-thick concrete pavers.)															
50,000 (6.3)		8.0	6.0	6.0	6.0	8.5	6.0	6.0	6.0	9.0	6.0	6.0	6.0	9.5	6.0	6.0	6.0
100,000 (6.8)		12.0	8.0	6.0	6.0	13.0	8.5	6.0	6.0	13.0	9.0	6.0	6.0	14.0	9.5	6.0	6.0
200,000 (7.4)		16.5	12.0	8.0	6.0	17.0	13.0	8.5	6.0	17.5	13.0	9.0	6.0	18.0	13.5	9.5	6.5
300,000 (7.8)		18.5	14.0	10.0	7.0	20.0	15.0	11.0	8.0	20.0	15.5	11.0	8.5	20.5	15.5	11.5	8.5
400,000 (8.1)		20.5	15.5	11.5	8.5	21.5	16.5	12.5	9.5	21.5	17.0	13.0	9.5	22.0	17.5	13.0	10.0
500,000 (8.3)		21.5	17.0	13.0	9.5	23.0	18.0	13.5	10.5	23.0	18.0	14.0	10.5	23.5	18.5	14.0	11.0
600,000 (8.5)		23.0	18.0	14.0	10.5	24.0	19.0	14.5	11.0	24.0	19.0	15.0	11.5	24.5	19.5	15.0	12.0
700,000 (8.6)		23.5	18.5	14.5	11.0	25.0	19.5	15.0	12.0	25.0	20.0	15.5	12.0	25.5	20.5	16.0	12.5
800,000 (8.8)		24.5	19.5	15.0	12.0	25.5	20.0	16.0	12.5	26.0	20.5	16.0	13.0	26.0	21.0	16.5	13.5
900,000 (8.9)		25.0	20.0	15.5	12.5	26.0	21.0	16.5	13.0	26.5	21.0	16.5	13.5	27.0	21.5	17.0	14.0
1,000,000 (9.0)		25.5	20.5	16.0	13.0	27.0	21.5	17.0	13.5	27.0	21.5	17.0	14.0	27.5	22.0	17.5	14.5

Note: Subbase thickness is calculated by dividing metric thicknesses in Table 4-9 by 25 and rounding to nearest 0.5 in.

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Table 4-10. Guidelines for Hydrologic Method Selection

	Factors for Consideration									*Models*			
Stormwater Management Criteria	*Local Hydrology*	*Available Storage Capacity*	*Aggregate Layer Porosity*	*Hydraulic Loading*	*Chemical Characteristics*	*Run-On Quality*	*Outfalls or Underdrains*	*Subgrade Infiltration*	*Storage Routing*	*Water Balance Estimate*	*Time-Step Method*	*Event-Based Hydrograph*	*Continuous Simulation*
Stormwater volume control	√	√	√	√				√		●	●	○	●
Water quality			√		√	√		√		○	●		○
Water thermal characteristics		√		√		√	√	√	√		●	●	●
Flood and/or peak flow control	√	√	√	√			√	√	√	●		●	●
Downstream erosion control	√	√	√	√			√	√	√	○		○	●
Infiltration and/or recharge targets	√	√	√	√			√	√		○	●		○
Ecosystem and habitat	√				√	√	√	√	√				●

Note: √ = Important factor to consider; ● = Typical application; ○ = Could be considered.

Table 4-11. Criteria for Assignment of Hydrologic Soil Groups (HSGs)

Depth to Water Impermeable Layer[a]	*Depth to High Water Table*[b]	K_{sat} *of Least Transmissive Layer in Depth Range*	K_{sat} *Depth Range*	*HSG*[c]
<50 cm (<20 in.)	—	—	—	D
50 to 100 cm (20 to 40 in.)	<60 cm (<24 in.)	>40.0 µm/s (>5.67 in./h)	0 to 60 cm (0 to 24 in.)	A/D
		>10.0 to ≤40.0 µm/s (>1.42 to ≤5.67 in./h)	0 to 60 cm (0 to 24 in.)	B/D
	≥60 cm (≥24 in.)	>1.0 to ≤10.0 µm/s (>0.14 to ≤1.42 in./h)	0 to 60 cm (0 to 24 in.)	C/D
		≤1.0 µm/s (≤0.14 in./h)	0 to 60 cm (0 to 24 in.)	D
	≥60 cm (≥24 in.)	>40.0 µm/s (>5.67 in./h)	0 to 50 cm (0 to 20 in.)	A
		>10.0 to ≤40.0 µm/s (>1.42 to ≤5.67 in./h)	0 to 50 cm (0 to 20 in.)	B
		>1.0 to ≤10.0 µm/s (>0.14 to ≤1.42 in./h)	0 to 50 cm (0 to 20 in.)	C
		≤1.0 µm/s (≤0.14 in./h)	0 to 50 cm (0 to 20 in.)	D
>100 cm (>40 in.)	<60 cm (<24 in.)	>10.0 µm/s (>1.42 in./h)	0 to 100 cm (0 to 40 in.)	A/D
		>4.0 to ≤10.0 µm/s (>0.57 to ≤1.42 in./h)	0 to 100 cm (0 to 40 in.)	B/D
		>0.40 to ≤4.0 µm/s (>0.06 to ≤0.57 in./h)	0 to 100 cm (0 to 40 in.)	C/D
		≤0.40 µm/s (≤0.06 in./h)	0 to 100 cm (0 to 40 in.)	D
	60 to 100 cm (24 to 40 in.)	>40.0 µm/s (>5.67 in./h)	0 to 50 cm (0 to 20 in.)	A
		>10.0 to ≤40.0 µm/s (>1.42 to ≤5.67 in./h)	0 to 50 cm (0 to 20 in.)	B
		>1.0 to ≤10.0 µm/s (>0.14 to ≤1.42 in./h)	0 to 50 cm (0 to 20 in.)	C
		≤1.0 µm/s (≤0.14 in./h)	0 to 50 cm (0 to 20 in.)	D
	>100 cm (>40 in.)	>10.0 µm/s (>1.42 in./h)	0 to 100 cm (0 to 40 in.)	A
		>4.0 to ≤ 10.0 µm/s (>0.57 to ≤1.42 in./h)	0 to 100 cm (0 to 40 in.)	B
		>0.40 to ≤4.0 µm/s (>0.06 to ≤0.57 in./h)	0 to 100 cm (0 to 40 in.)	C
		≤0.40 µm/s (≤0.06 in./h)	0 to 100 cm (0 to 40 in.)	D

[a] An impermeable layer has a K_{sat} less than 0.01 µm/s [0.0014 in./h] or a component restriction of fragipan; duripan; petrocalcic; orstein; petrogypsic; cemented horizon; densic material; placic; bedrock, paralithic; bedrock, lithic; bedrock, densic; or permafrost.

[b] High water table during any month during the year.

[c] Dual HSG classes are applied only for wet soils (water table lower than 60 cm or 24 in.). If these soils can be drained, a less restrictive HSG can be assigned, depending on the K_{sat}.

Source: NRCS (2009).

Table 4-12. Summary of Inputs and Outputs for Each Infiltration Design Type

	Inputs		*Outputs*	
Design Type	*Direct Rainfall* (P)	*Run-On from Contributing Areas* (R)	*Infiltration into Subgrade* (I)	*Underdrain Discharge* (Q_U)
Full infiltration	Yes	Possible	Yes	No (*Z* = 0)
Partial infiltration	Yes	Possible	Yes	Yes (*Z* between 0 and 1)
No infiltration	Yes	Possible	No	Yes (*Z* = 1)

Chapter 5

CONSTRUCTION GUIDELINES

5.1 CONSTRUCTION STEPS

PICP construction follows the steps listed following. A guide construction specification is provided in Appendix B and should be modified to project conditions.

- Schedule a preconstruction meeting to be attended by owner's on-site representative, construction trades, and material vendors.
- Identify and protect any underground utilities.
- Follow construction sequencing plan per contract documents to keep PICP materials free from sediment and contamination. Utilize erosion and sediment control measures.
- Install any underground utilities.
- Excavate design pavement depth to subgrade (protect existing utilities, tree roots, buildings and traditional pavement structures, etc.).
- Prepare subgrade as specified.
- Install geosynthetics and underdrains if required in the design.
- Place and compact the aggregate subbase; test deflections with a lightweight deflectometer
- Install curbs or other edge restraints.
- Place and compact the aggregate base; test deflections with a lightweight deflectometer.
- Place and screed the bedding layer. Do not compact at this time.
- Install pavers manually or with mechanical installation equipment.
- Fill the paver joints and sweep the surface clean.
- Compact the pavers and the bedding layer together.
- Fill joints with additional jointing stone as needed, sweep the surface clean and compact again.
- Confirm final elevations of the pavement surface at 6 mm (1/4 in.) above adjacent curbs, utility structures, etc.
- Return 3 to 6 months after completion of construction to inspect pavement and refill joints with aggregate, as needed.

5.2 PRECONSTRUCTION MEETING

Project specifications should require a preconstruction meeting. The purpose of the preconstruction meeting is to discuss methods of accomplishing all phases of the construction operation, contingency planning, and standards of workmanship. The general contractor typically provides the meeting facility, meeting date, and time. Sufficient time should be available after the meeting to allow for the construction and approval of the mock-up and subsequent production of the paving units. Representatives from the following entities should be present:

- Owner's representative
- General contractor superintendent
- All related subcontractor representatives (i.e., excavator, PICP installer, etc.)
- Concrete paving unit manufacturer's representative
- Testing laboratory representative(s), and
- Engineer or design representative.

The following items should be discussed and determined:

- Walk through site with builder/contractor/subcontractor to review erosion and sediment control plan/stormwater pollution prevention plan or "SWPPP."
- Determine when PICP is built in project construction sequence; before or after building construction, and measures for PICP protection and surface cleaning.
- Identify Aggregate material stockpile locations (hard surface or on geotextile).
- Review test panel (mock-up) location, dimensions and criteria for acceptance.
- Contractor's methods for keeping all materials free from sediment during storage, placement, and on completed areas.
- Contractor's methods for checking slopes, surface tolerances, and elevations.
- Contractor's methods and procedures to accommodate unexpected field conditions
- Paver unit delivery method(s), timing, storage location(s) on the site, staging, paving start point(s), and direction(s).
- Anticipated daily paving production and actual record.
- Diagrams of paving laying/layer pattern, stitching requirements, and joining layers
- Monitoring/verifying paver dimensional tolerances in the manufacturing facility and on-site if the concrete paving units are mechanically installed.
- Testing intervals for aggregates, edge restraints, and for the concrete paving units.
- Method(s) for tagging and numbering concrete unit paving packages delivered to the site.
- Testing lab location, test methods, report delivery, contents, and timing.

- Contractor's quality control and assurance methods and reporting. This should include at least a crew leader or foreman on the job site that holds a certificate of completion in the Interlocking Concrete Pavement Institute PICP specialist course.
- Engineer inspection intervals and procedures for correcting work that does not conform to the project specifications.

5.3 EROSION AND SEDIMENT CONTROL

Sediment management is one of the most critical aspects of PICP construction. Inappropriate or lack of planning, construction staging and/or erosion and sediment control techniques can introduce unwanted low permeability sediments (silts and clays) into the excavation, aggregates or onto the paver surface. All can reduce the subgrade or surface infiltration rates, and reduce the hydrologic effectiveness and functionality of the pavement system.

The project erosion and sediment control plan should identify procedures to prevent and divert sediment from entering the permeable pavement system during construction. Sediment should be kept completely away from aggregates stored on site as well as the PICP. In some cases, it may be necessary to construct PICP before other soil-disturbing construction is completed. The site should be inspected to identify potential sediment sources. Exposed areas should be stabilized prior to construction using approved methods.

Permeable pavements typically require enhanced erosion and sediment control procedures to protect their surfaces from damage/contamination during construction. While preventing erosion, such practices maintain infiltration capabilities, prevent tracking of soil and construction materials onto the permeable pavement, and so on. Other practices can be considered, such as keeping muddy construction equipment away from the PICP, installing silt fences, staged excavation, vehicle tire/track washing stations, and temporary drainage swales that divert runoff away from the area.

As noted in Table 3-1, PICP built early in the construction sequencing may clog with sediment from construction site runoff/run-on or vehicles. The surface may require remedial cleaning prior to project acceptance. A detailed erosion and sediment control plan that diverts sediment sources away from the permeable pavement is essential for owner acceptance at the end of construction, as well as for long-term performance of PICP. Sediment management procedures could include the following:

- Establish an appropriate construction staging plan. The staging plan is an integral point of sediment management during construction.

- Develop a hierarchical plan which focuses on erosion prevention as a first approach, followed by sediment management. By preventing erosion, the management of sediment is minimized and the risk of contamination of the pavement system can be mitigated.
- Identify access routes for delivery and construction vehicles (see items 1 through 4 following)
- Vehicle tire/track washing station (if specified in Erosion and Sediment plan/SWPPP) location/ maintenance.
- All runoff diverted away from the completed excavated area.
- Temporary soil stockpiles should be protected from run-on, runoff from adjacent areas and from erosion by wind.
- Ensure linear sediment barriers (if used) are properly installed, free of accumulated litter, and built up sediment less than 1/3 the height of the barrier.
- No runoff enters PICP until soils are stabilized in the area draining to the PICP.
- Should the excavated area be used as a temporary sediment trap, the invert of the temporary sediment trap should be 300 mm (12 in.) or more above the top of the permeable pavement subgrade. The erosion control plan designer could consider the use of an impermeable liner to protect the design infiltration rate of the subgrade. Additional excavation to the pavement subgrade design grades should not occur until the site has been stabilized and the sediment trap is removed from use. Additional excavation below the pavement subgrade design grades may still be necessary afterward to ensure the design infiltration rate is achieved.

In order to restrict erosion and sediment deposition from construction vehicles, the options below should be considered in the project planning stages and appropriate one(s) included in the project specifications:

1. Construct the aggregate subbase and base and protect the surface of the base aggregate with geotextile and an additional 50 mm (2 in.) thick layer of open graded aggregate over the geotextile. Thicken this layer at transitions to match elevations of adjacent pavement surfaces subject to vehicular traffic. A similar, more expensive approach can be taken using a temporary asphalt wearing course rather than the additional base aggregate and geotextile. When construction traffic has ceased and adjacent soils are vegetated or stabilized with erosion control mats, remove geotextile and contaminated aggregate (or the asphalt) and install the remainder of the PICP system per the project specifications.
2. Install the PICP first and allow construction traffic to use the finished PICP surface. When construction traffic has ceased and adjacent soils are stabilized with vegetation or erosion control mats, clean the PICP surface and joints with a vacuum machine capable of removing stone to an approximate 25 mm (1 in.) depth within the joints. Vacuum a test area and inspect the joints where stone is removed to be sure there are no visible traces of sediment on the stone remaining

in the joints below. If it is visible, then vacuum out jointing stones until no sediment is present. Fill the joints with clean stones and sweep the PICP surface clean.

3. Protect finished PICP system by covering the surface with a geotextile and a minimum 50 mm (2 in.) thick ASTM No. 8 open-graded aggregate layer. This aggregate layer and geotextile are removed upon project completion and when adjacent soils are stabilized with vegetation or erosion control mats. The PICP surface is then swept clean and inspected. If there are visible traces of sediment in the joints remaining, then vacuum out jointing stones until no sediment is present. Fill the joints with clean stones and sweep the PICP surface clean.
4. Establish temporary road(s) for site access that do not allow construction vehicle traffic to ride over and contaminate the PICP base materials and/or surface with mud and sediment. Other trades on the jobsite need to be informed on using temporary road(s) and staying off the PICP. The temporary road is removed upon completion of construction and before opening of the PICP surface to traffic.

5.4 CONSTRUCTION INSPECTION CHECKLIST

The following provides a construction checklist for project use. The engineer should edit according to specific project requirements.

Preconstruction meeting

- See Section 5.2

Erosion and Sediment Control

- See Section 5.3

Excavation

- Utilities located and marked by local service.
- Excavated area marked with paint and/or stakes.
- Excavation size and location conforms to plan.

Adjacent Structures

- Verify appropriate treatment or offset from foundation walls.
- Verify grades and flow paths of adjacent surfaces discharging to the PICP system are consistent with designs.
- Verify appropriate treatment of adjacent traditional pavement areas.
- Verify appropriate treatment or adjustment of utilities.
- Verify utility collar installation.

Water supply

- Verify offset from water supply wells per local requirements.

Soil subgrade

- Subgrade should be free from obstructions such as large stones, stumps, organic material, etc.
- Soil compacted to specifications (if required) and field tested with density measurements per specifications.
- For full or partial-infiltration designs, verify the design infiltration rate.
- Address any unanticipated conditions in the subgrade such as unsuitable soils or bedrock encountered during construction.
- No groundwater seepage or standing water, if so, dewatering and a dewatering permit may be required. The bottom of the permeable pavement system should be at an elevation of at least 0.6 to 1 m (2 to 3 ft) from the annual high water table.
- Soil infiltration (permeability) test reports.
- Laboratory report(s) on compacted Proctor density and subgrade density testing reports if compaction is required.

Geosynthetics (if specified)

- Verify that delivered geosynthetic materials meet specifications including geotextiles, geogrids and/or geomembranes.
- Placement, down slope overlap and fastening to conform to specifications and drawings. No wrinkles, pulled taught and staked.
- Sides of excavation covered with geotextile prior to placing aggregate base/subbase.
- Protect geosynthetics during construction. No tears or holes.
- Use geotextiles as necessary to protect geomembranes below the reservoir layer for no-infiltration systems.
- Placement, field welding, and seals at pipe penetrations for geomembranes done per specifications and for water and soil tightness.

Drain pipes/cleanouts/observations wells

- Size, perforations, locations, slope, and outfall meet specifications and drawings.
- Ensure all connections are made with manufacturer approved fittings.
- Verify elevation of outlet pipes and inverts.
- Verify location and protection of observation well(s) and cleanouts while installing base, subbase, and wearing surface. Adjust caps flush to grade.

Subbase, base, bedding, and jointing aggregates

- Sieve analysis from quarry conforms to specifications.

- Store materials on hard surfaces or geotextiles to keep them sediment-free. Cover and or wash materials as necessary.
- Spread (not dumped) with a front-end loader to avoid aggregate segregation. Use clean trucks to haul open graded aggregate and minimize fines.
- Placement, compaction equipment and methods, compacted lift thickness, lightweight deflectometer testing methods, final thickness for the subbase and base, and surface tolerances for all aggregate layers meet the specifications and drawings.
- Excess jointing aggregate is swept from the surface before compaction. Excess jointing material is used as necessary to fill joints after compaction including during any warranty period.

Edge restraints

- Elevation, placement, and materials to meet specifications and drawings.

Permeable interlocking concrete pavers

- Meet ASTM/CSA standards (as applicable) per manufacturer's test results.
- Elevations, slope, laying pattern, joint widths, and placement/compaction meet drawings and
- Specifications.
- No cut pavers subject to vehicular traffic shall be less than one third of a whole paver.
- Verify that the pavers beyond 2 m (6 ft) of the laying face are fully compacted at the completion of each day, starting from one side of the installation.
- Verify the surface tolerance of compacted pavers deviate no more than ±10 mm (3/8 in.) under a 3 m (10 ft) long straight edge in all directions.

Final inspection

- Surface swept clean. No damaged unit pavers.
- Elevations and slope(s) conform to drawings.
- PICP is entirely surrounded with edge restraints.
- Surface elevation of pavers 6 to 10 mm (1/4 to 3/8 in.) above adjacent drainage inlets, concrete collars, or channels.
- Lippage: no greater than 3 mm (1/8 in.) difference in height between adjacent pavers.
- Bond lines for paver courses: ±15 mm (±½ in.) over a 15 m (50 ft) string line.
- Stabilization of soil in areas draining into permeable pavement (min. 6 m or 20 ft wide vegetative strips are recommended).
- Confirm the function of drainage swales or storm drain inlets for emergency overflow.
- Runoff from non-vegetated soil diverted away from PICP surfaces.

- Test surface for infiltration rate per specifications using ASTM C1781 *Standard Test Method for Surface Infiltration Rate of Permeable Unit Pavement Systems*; minimum 2,500 mm/h (100 in./h) recommended.

Chapter 6

MAINTENANCE GUIDELINES

6.1 PAVEMENT MAINTENANCE

All pavements require maintenance including PICP. Routine maintenance will prolong pavement life and increase performance, resulting in a cost-effective system accommodating traffic while assisting in achieving stormwater objectives. Proactive maintenance is more effective than reactive maintenance. As the pavement ages, it may be necessary to treat localized areas to restore the pavement surface condition. This may include removal of paving units, leveling or addition of new aggregate materials, removal and replacement of jointing material, and so on. Edge restraints should be inspected to ensure that they are performing their required function. Outlet drains and observation wells should be inspected to confirm continued drainage from the pavement structure. Areas upslope of the PICP should be examined for potential sources of contaminants that may reduce system permeability and for grade changes that may result in unintended ponding or flow adjacent to or around the PICP system and for grade changes that may result in unintended ponding or flow adjacent to or around the PICP system.

Maintenance for PICP systems is similar to conventional pavements except that winter sand should not be used for traction purposes. Sand will enter the joints between the pavers and potentially reduce the system permeability. Deicing chemicals such as sodium or calcium chloride can be used, but the quantity needed will likely be less for PICP systems than for conventional pavements due to the thermal gain and the ability of PICP to infiltrate meltwater and the associated reduction in ice formation for many installations.

A maintenance plan should be developed and followed for PICP. The plan should include documentation of key design features of the system, operational constraints (e.g., restrict use of winter sand, maintain overflow and outflow features, monitor observation wells, etc.), inspection schedules and checklists, maintenance procedures, rehabilitation activities and timing, and others. Personnel responsible for the maintenance and operation of the PICP should be identified and provided with the maintenance plan. The maintenance plan should be reviewed and modified based on actual use and operation of the facility.

The following sections provide guidance for inspections, and routine, remedial and winter maintenance activities.

6.2 INSPECTION ACTIVITIES

PICP inspections should be completed 1 to 2 times annually (preferably shortly after a storm event). Inspection tasks may include the following:

- Review maintenance and operations records and incidences to determine if there have been any issues.
- Document general site features (take photographs, etc.).
- Note any surface contamination or clogging. Define area by area measurements as a percentage of the overall surface area.
- Note obvious sources of surface contaminants.
- Identify the extent and severity of any damage or deficiencies (e.g., settlement, ponding, cracked pavers, etc.).
- Identify any changes in adjacent land use that may impact contributing area runoff for potential sources of contaminants that may reduce system permeability.
- Inspect vegetation around PICP perimeter for cover and soil stability.
- Edge restraints should be inspected to ensure that they are performing their required function.
- Underdrains should be checked to ensure that they are still draining water from the pavement structure.
- Observation wells should be checked to confirm the maximum allowable storage time.
- If the observed performance of the pavement indicates a significant reduction in permeability from the last inspection, complete infiltration testing in accordance with local or project specific requirements. In the absence of site specific requirements, use the test method outlined following.

6.3 INFILTRATION TESTING

Surface infiltration should be measured using ASTM C1781 *Standard Test Method for Surface Infiltration Rate of Permeable Unit Pavement Systems* (ASTM 2015). Fig. 6-1 illustrates the C1781 test apparatus. A 300 mm (12 in.) diameter plastic or metal ring is secured to the PICP surface with plumber's putty. A known mass of water is poured at a constant rate and timed. The test method includes a formula for calculating the infiltration rate. The minimum recommended acceptable infiltration for new PICP is 2,500 mm/h (100 in./h). This initial value accommodates some sedimentation and surface clogging that may occur over time. The recommended in-service PICP surface infiltration should also not be less than 250 mm/h (10 in./h).

C1781 can be used for assessing when surface vacuum cleaning might be required. The surface infiltration test results from C1781 are comparable with test results on pervious concrete or porous asphalt using ASTM C1701 *Standard Test Method for Infiltration Rate of In Place Pervious Concrete* (ASTM 2017d).

The results of the inspection should be documented and used to assist in updating the maintenance plan for the PICP system. The information should be used to assist in predicting future maintenance needs and be part of an overall management system for the pavement. Based on the results of the inspection, it may be appropriate to conduct maintenance work.

6.4 ROUTINE MAINTENANCE

The following provides a checklist for PICP routine maintenance:

- Perform a general cleaning of the pavement surface to remove debris and sediment when the pavement is dry. Coarse sediment may be removed by a conventional street sweeper, but care should be taken to ensure that smaller sediment does not just end up being spread over a larger area. Regenerative air sweepers are preferred as they prevent further spreading of sediment, and are effective in removing sediment from between the upper joints. Vacuum machine cleaning may be necessary to remove the sediment from within clogged or compacted joints and to restore permeable pavement surface infiltration capacity. For low sediment and low use areas, the maintenance requirements may be less frequent.
- Repair/replant vegetative cover for areas upslope from the PICP.
- Replace any damaged or broken pavers.
- Replenish aggregate in joints if more than 13 mm (1/2 in.) is exposed below the chamfer bottoms on the unit pavers.
- Repair all paver surface deformations exceeding 13 mm (1/2 in.).
- Repair pavers offset by more than 6 mm (1/4 in.) above/below adjacent units or curbs, inlets etc.
- Clean and flush underdrain systems using cleanouts or other access points.
- Clean drainage outfall features to ensure free flow of water and outflow.
- Clean and remove sediment from any pretreatment devices or sediment traps.

6.5 REMEDIAL MAINTENANCE

- If pavement appears “clogged” through visual observation or standing water after a rain event, test the pavement using ASTM C1781 to determine surface infiltration rate in that location. Vacuum to remove surface sediment and soiled aggregate (typically 13 to 25 mm or ½ to 1 in.

deep), refill joints with clean aggregate, sweep surface clean and test again per C1781 to minimum 50% increase or minimum 250 mm/h (10 in./h) infiltration rate.

- Repair and/or reinstate damaged edge restraints. May require removal and reinstatement of adjacent paving units.
- Replace cracked paver units impairing surface structural integrity.
- Repair localized settlement and rutted pavement areas.
- Regrade or modify changed grades for areas upslope of the PICP system where unintended ponding or informal flow paths around the PICP system may have developed.
- Repair outflow features, piping, energy dissipaters, erosion protection systems, pre-treatment systems, and so on.
- Repair localized settlement.
- Regrade or modify changed grades for areas upslope of the PICP system where unintended ponding or informal flow paths around the PICP system have developed.

6.6 WINTER MAINTENANCE

PICP facilitates continued surface drainage, subgrade infiltration, and pollutant reductions during the winter months. PICP systems with standing water within the base or subbase will freeze during prolonged freezing temperatures typical to northern climates. In the late winter, PICP resumes infiltration and pollutant reductions prior to temperatures achieving levels consistently above freezing.

Prohibit/discourage the use of winter sand for traction, which may clog the pavement. Limit, or if possible, eliminate winter sand altogether. Procedures for snow and ice removal are similar to those for conventional pavements. The need for deicing chemicals however, may be reduced as part of removing snow and ice. Prohibit deicing chemicals use if water is to be captured and reused. If not removed in the spring, the accumulation of fine sand during the winter may clog permeable pavement surfaces resulting in reduced surface infiltration. Other winter maintenance guidance includes the following:

- Avoid the use of winter sand for traction. Alternatively, consider the use of aggregate similar to the jointing material. If traction materials are used, vacuum clean the surface in the early spring.
- Paver manufacturer recommendations should be consulted regarding the type and application rates of deicing chemicals.
- Monitor ice on surface and apply deicers as needed. A reduction in deicer use may occur compared to that used on impervious pavements.

- When applying a deicer, many paver manufacturers recommend using sodium chloride. Other types of deicer should be carefully evaluated to minimize damage to the pavers, particularly products that contain a blend of chemicals.
- Remove snow with standard plow/snow blowing or sweeping equipment. Use rubber/plastic tipped plow blades if surface abrasion is not desired. Deposit snow off the PICP surface and onto vegetated surfaces.
- The size/weight of winter maintenance equipment should not exceed the structural capacity of the pavement.

Figure 6-1. ASTM C1781 Test Apparatus to Measure Surface Infiltration.

Source: ICPI (2017c); reproduced with permission from Interlocking Concrete Pavement Institute.

APPENDIX A

DESIGN EXAMPLES

A.1 STRUCTURAL DESIGN EXAMPLE

Step 1—Determine Structural Design Input Parameters. A permeable interlocking concrete pavement is required for a two-lane entrance roadway to a recreational area. Input data include the following:

- Location = Ottawa, Canada
- Pavement service life = 25 years
- Average annual daily traffic (AADT) = 2,085 vehicles/day
- Directional distribution = 0.5
- Lane distribution = 1
- Commercial vehicle percent = 0.1
- Commercial vehicle load factor = 0.76
- Traffic days/year = 365
- Traffic annual growth rate = 0.02
- Initial design serviceability = 4.2
- Terminal design serviceability = 2.5
- Design reliability = 80%
- Standard deviation = 0.44
- Subgrade = Silty sand
- Subgrade resilient modulus (wet) = 48 MPa (6,960 lb/in.^{2})
- Subgrade permeability = 14 mm/day (0.5 in./day)
- Average precipitation per year = 869 mm
- Average days where precipitation > 0.1 mm = 141
- Average days where precipitation > 14 mm/day = 30
- Surface = 80 mm (3.125 in.) pavers + 50 mm (2 in.) bedding layer

Step 2—Determine Design ESALS. Use the AADT, directional distribution (DD), lane distribution (LD), commercial vehicle percentage (CV), heavy vehicle factor (TF), number of days per year that the pavement is subjected to heavy vehicles (DY), and the traffic annual growth rate (GR) to determine the design ESALs for a pavement service life of 25 years.

From Eq. (4-1):

$$\mathrm{ESALs} = \mathrm{AADT} \times \mathrm{DD} \times \mathrm{LD} \times \mathrm{CV} \times \mathrm{TF} \times \mathrm{DY} \times \left[\frac{(1+\mathrm{GR})^{\mathrm{DL}}-1}{\mathrm{GR}}\right] \qquad (4\text{-}1)$$

$$\mathrm{ESALs} = 1{,}125 \times 0.5 \times 1 \times 0.76 \times 0.1 \times 365 \times [((1+0.02)^{25} - 1)/0.02] = 499{,}793$$

Step 3—Determine the Required Pavement Structure. Use Table 4-8. Find the 500,000 design ESALs and (wet) subgrade resilient modulus of 48 MPa (6,960 lb/in.^{2}) within the column labeled average 11 to 30 days per year with water in the subbase (i.e., precipitation exceeds the infiltration capacity of the subgrade). Find the subbase thickness at 280 mm (11 in.).

Design pavement structure =

80 mm (3 1/8 in.) permeable interlocking concrete pavers
50 mm (2 in.) ASTM No. 8 bedding layer
100 mm (4 in.) ASTM No. 57 base
280 mm (11 in.) ASTM No. 2 subbase
Total thickness = 510 mm (20 in.)

A.2 NO-INFILTRATION DESIGN EXAMPLE

Step 1—Assess Site Conditions. A parking lot is being designed in an urbanized area where storm drains have limited capacity to convey runoff because of increases in impervious surfaces. Input data include the following:

- PICP area (A_P) = 4,000 m^2 (1 ac). $A_P \approx A_I$.
- Run-on (A_C) from an 8,000 m^2 (2 ac) asphalt parking lot needs to be accommodated. Runoff coefficient (C) for asphalt parking lot is 0.98.
- Soil borings reveal that the seasonal high water table is 3 m (10 ft) below the subgrade.
- The subgrade USCS soil classification is organic clay with low infiltration capabilities and high swelling potential.
- The porosity (n) in the ASTM No. 57 base and ASTM No. 2 subbase provided by the local quarry are 40% or 0.40.
- Local regulations require this site to capture and discharge, under controlled conditions, all rainfall and runoff from a two-year, 24 h (T_S) design storm (P) of 0.076 m (3 in. or 0.25 ft).

- The permitted discharge rate from the site is 0.01 $m^3/s/ha$ (0.15 cubic feet per second per acre [($ft^3/s/ac$)]. An orifice plate is required on the underdrain to verify compliance.
- A maximum 48 h drawdown (T_D) is required.
- The project is not close to building foundations, nor are there any wells in the area.

Step 2—Calculate the Diameter of the Underdrain. Calculate the run-on depth from the contributing area (R) for use in the equation following:

$$R = P \times C = 0.076 \times 0.98 = 0.07448$$

Determine the maximum potential driving head ($H_{o\text{–max}}$) using Eq. (4-17). Start off with an assumed pipe radius (r_{pipe}) of 50 mm (2 in.). The pipe is to be located within a trench below the base or subbase, and it is to have 50 mm (2 in.) of pipe surround on all sides.

$$H_{o-\max} = \frac{(PA_P + RA_C)}{A_P * n} + r_{\text{pipe}} + \text{pipe cover}$$

$$\begin{aligned} H_{o\text{–max}} &= (0.076 \times 4{,}000 + 0.07448 \times 8{,}000)/(4{,}000 \times 0.4) + 0.05 + 0.05 \\ &= (304 + 596)/(1{,}600) + 0.05 \\ &= 0.5625 + 0.10 \\ &= 0.6625 \text{ m } (26.08 \text{ in.}) \end{aligned}$$

The maximum permitted discharge from the site is 0.0127 m^3/s (0.45 ft^3/s), which is based on the permitted discharge rate of 0.15 $ft^3/s/ac$ multiplied by the total contributing and PICP area, 12,000 m^2 (3 ac).

The orifice plate equation from Eq. (4-15) is used to calculate the cross-sectional area of the orifice (A) and in turn the pipe radius. Inputs into the equation include the following:

Flow rate (Q_U) = 0.0127 m^3/s (0.45 ft^3/s)

Coefficient of discharge (C_D) = 0.80 for an orifice tube.

Acceleration (g) = 9.80665 m/s^2 (32.174 ft/s^2)

Differential head measured (Δh) = $H_{o\text{–max}}$ = 0.6625 m (26.08 in.)

$$Q_U = C_D \times A \times \sqrt{2g\Delta h}$$

Rearranging the equation gives us the following:

$$\begin{aligned} A &= 0.0127/(0.8 \times \text{sqrt}\ (2 \times 9.80665 \times 0.6625)) \\ &= 0.0127/(0.8 \times 3.605) \\ &= 0.0044 \text{ m}^2 \end{aligned}$$

$$\begin{aligned} \text{Pipe radius} &= \text{sqrt}\ (A/\pi) \\ &= \text{sqrt}\ (0.0044/3.14159) \end{aligned}$$

$$= 0.0374 \text{ m}$$

The pipe radius of 0.0374 m is not close enough to the original assumption of 0.05 m; therefore, recalculate using $R = 0.0389$ m.

$$H_{o\text{–max}} = 0.5625 + 0.038 + 0.05 = 0.6505 \text{ m}$$

$$A = 0.00444 \text{ m}^2$$

$$\text{Pipe radius} = 0.0376 \text{ m (within acceptable range)}$$

Step 3—Compute the Flow Rate from the Pipe to Be Used for the Water Balance. The outlet flow rate through the underdrains (Q_U) needs to be constant to conduct a preliminary estimate of required water storage volume. Using the maximum flow rate out of the pipe for Q_U would overestimate the actual pipe discharge in all but extreme conditions. A representative percentage of the maximum flow rate should be used in the modeling for Q_U based on engineering judgment. In this example, a conservative value of Q_U is calculated using a differential head (Δh) equal to the pipe radius of 0.038 m.

$$Q_U = 0.8 \times (\pi \times 0.038^2) \times \text{sqrt}\,(2 \times 9.80665 \times 0.038)$$

$$= 0.8 \times (0.00454) \times (0.8633)$$

$$= 0.0033 \text{ m}^3/\text{s}$$

$$= 11.29 \text{ m}^3/\text{h}$$

Step 4—Calculate the Maximum Volume of Water to Be Stored in the Pavement Base (V_W).

From Eq. (4-14): $V_W = P(A_P) + R(A_C) - Q_U T_S$

$$V_w = (0.076 \times 4{,}000) + (0.07448 \times 8{,}000) - 11.29 \times 24$$

$$= 304 + 596 - 271$$

$$= 629 \text{ m}^3$$

Step 5—Calculate the Required Depth of the Base or Subbase. Calculate the hydraulic depth required using Eq. (4-13):

$$\text{Hydraulic depth} = \frac{V_W}{n * A_P}$$

$$= 529/(0.4 \times 4{,}000)$$

$$= 0.393 \text{ m (15.48 in.)}$$

Assuming that both the base and subbase are being used to provide hydraulic capacity, the hydraulic cross section would be 100 mm (4 in.) of base and 293 mm (11.48 in.) of subbase. This result is acceptably close to the structural design result of 100 mm (4 in.) thick base and 280 mm (11 in.) thick subbase. Round the subbase thickness up to 300 mm (or 12 in.) for construction convenience.

A.3 FULL-INFILTRATION DESIGN EXAMPLE

Step 1—Assess Site Conditions. A parking lot is being designed in an urbanized area where storm drains have limited capacity to convey runoff because of increases in impervious surfaces. Input data include the following:

- PICP area (A_P) = 4,000 m^2 (1 ac). $A_P \approx A_I$.
- Run-on (A_C) from an 8,000 m^2 (2 ac) asphalt parking lot needs to be accommodated. Runoff coefficient (C) for an asphalt parking lot is 0.98.
- Soil borings reveal that the seasonal high water table is 3 m (10 ft) below the subgrade.
- The subgrade USCS soil classification is sand. Infiltration was field-tested at 0.21 m/h (8.26 in./h). Although this was the tested rate, the designer is taking a conservative position for the design infiltration rate (I) by assuming it at half of the field result or 0.105 m/h (4.18 in./h). This approach accounts for the potential loss of permeability from construction, soil compaction, and soil subgrade clogging over time.
- The porosities (n) in the ASTM No. 57 base and ASTM No. 2 subbase provided by the local quarry are 40% or 0.40.
- Local regulations require this site to capture and, when possible, infiltrate all rainfall and runoff from a two-year, 24-h (T_S) design storm (P) of 0.076 m (3 in. or 0.25 ft).
- The permitted discharge rate from the site is 0.01 m^3/s/ha [0.15 cubic feet per second per acre (ft^3/s/ac)]. An orifice plate is required on the underdrain to verify compliance.
- A maximum 48-h drawdown (T_D) is required.
- The project is not close to building foundations, nor are there any wells in the area.

Step 2—Calculate the Maximum Volume of Water to Be Stored in the Pavement Base (V_w). From Eq. (4-19),

$$V_W = P(A_P) + R(A_C) - I(T_S)A_I$$

$$V_W = (0.076 \times 4{,}000) + (0.98 \times 0.076 \times 8{,}000) - (0.105 \times 24 \times 4{,}000)$$

$$= 304 + 596 - 10{,}080$$

$$= -9{,}180.16 \text{ m}^3 \ (94{,}593 \text{ ft}^3)$$

As the infiltration rate of the soil exceeds the overall inputs, there is a negative water balance; therefore, the base and subbase thickness from the structural analysis governs the design.

A.4 PARTIAL-INFILTRATION DESIGN EXAMPLE

Step 1—Assess Site Conditions. A parking lot is being designed in an urbanized area where storm drains have limited capacity to convey runoff because of increases in impervious surfaces. Input data include the following:

- PICP area (A_P) = 4,000 m^2 (1 ac). $A_P \approx A_I$.
- Run-on (A_C) from an 8,000 m^2 (2 ac) asphalt parking lot needs to be accommodated. Runoff coefficient (C) for an asphalt parking lot is 0.98.
- Soil borings reveal that the seasonal high water table is 3 m (10 ft) below the subgrade.
- The subgrade USCS soil classification is sandy clay. Infiltration was field-tested at 4.6 mm/h (0.18 in./h). Though this was the tested rate, the designer is taking a conservative position for the design infiltration rate (I) by assuming it at half of the field result or 2.3 mm/h (0.09 in./h). This approach accounts for the potential loss of permeability from construction, soil compaction, and soil subgrade clogging over time.
- The porosities (n) in the ASTM No. 57 base and ASTM No. 2 subbase provided by the local quarry are 40% or 0.40.
- Local regulations require this site to capture and, when possible, infiltrate all rainfall and runoff from a two-year, 24 h (T_S) design storm (P) of 0.076 m (3 in. or 0.25 ft).
- The permitted discharge rate from the site is 0.01 m^3/s/ha (0.15 cubic feet per second per acre [(ft^3/s/ac)]. An orifice plate is required on the underdrain to verify compliance.
- A maximum 24 h (1 day) drawdown (T_D) is permitted.
- The project is not close to building foundations, nor are there any wells in the area.

Step 2—Determine the Pipe Elevation (PE). Eq. (4-21) is used to determine the volume of water that can remain below the outlet invert for infiltration (infiltration storage depth), which is regulated by the amount of water that can infiltrate into the subgrade within the maximum allowable post-rainfall storage time (T_D). As noted in Section 4.2.1, use a drawdown that is below the maximum permitted to account for the potential post-rainfall storage time that water is above the pipe elevation. In this example, T_D is reduced by 25% (based on engineering judgment).

$$\text{PE} = \frac{I * T_D}{n}$$

$$\text{PE} = 0.0023 \times (24 \times 0.75)/0.4$$

$$= 0.1035 \text{ m (4 in.)}$$

Step 3—Calculate the Underdrain Elevation Factor (Z). The underdrain elevation factor (Z) represents the percentage of time that underdrain outflow occurs during the storm event. Use Eq. (4-23) to calculate how long it takes for the water level to reach the pipe elevation (PE), then use Eq. (4-24) to calculate *Z*. Remember that two of the assumptions made to allow for a simplified analysis are that “underdrains go instantaneously from no flow to full pipe flow conditions” and “the outlet flow rate through the underdrains is constant.”

$$T = \frac{\text{PE} \ * A_P * \ T_S * n}{PA_P + RA_C - IT_S A_I}$$

and

$$Z = 1 - \frac{T}{T_S}$$

Calculate the run-on depth from the contributing area (*R*) for use in the equation.

$$R = P \times C$$
$$= 0.076 \times 0.98$$
$$= 0.07448$$
$$T = (0.1035 \times 4{,}000 \times 24 \times 0.4)/(0.076 \times 4{,}000 + 0.07448 \times 8{,}000 - 0.002286 \times 24 \times 4{,}000)$$
$$= (3{,}974.4)/(304 + 596 - 220)$$
$$= 3{,}974.4/680$$
$$= 5.84 \text{ h}$$
$$Z = 1 - 5.84/24$$
$$= 0.757$$

Step 4—Calculate the Diameter of the Underdrain. Determine the maximum potential driving head ($H_{o\text{–max}}$) using Eq. (4-18). Start off with an assumed pipe radius (r_{pipe}) of 50 mm (2 in.).

$$H_{o-\max} = \frac{(PA_P + RA_C - IT_S A_P)}{(A_P * n)} - \text{ PE} - \ r_{\text{pipe}}$$

$$H_{o\text{–max}} = (0.076 \times 4{,}000 + 0.07448 \times 8{,}000 - 0.0023 \times 24 \times 4{,}000)/(4{,}000 \times 0.4) - 0.1035 - 0.05$$
$$= (304 + 596 - 220)/(1{,}600) - 0.1535$$
$$= 0.424 - 0.1535$$
$$= 0.2705 \text{ m (10.65 in.)}$$

The maximum permitted discharge from the site is 0.0127 m^3/s (0.45 ft^3/s), which is based on the permitted discharge rate of 0.15 $ft^3/s/ac$ multiplied by the total contributing and PICP area, 12,000 m^2 (3 ac).

The orifice plate equation from Eq. (4-15) is used to calculate the cross-sectional area of the orifice (*A*) and in turn the pipe radius. Inputs into the equation include the following:

Flow rate (Q_U) = 0.0127 m^3/s (0.45 ft^3/s)

Coefficient of discharge (C_D) = 0.80 for an orifice tube

Acceleration (g) = 9.80665 m/s^2 (32.174 ft/s^2)

Differential head measured (Δh) = $H_{o\text{–max}}$ = 0.2705 m (10.7 in.)

$$Q_U = C_D \times A \times \sqrt{2g\Delta h}$$

Rearranging the equation gives us the following:

$$A = 0.0127/(0.8 \times \text{sqrt}\,(2 \times 9.80665 \times 0.2705))$$
$$= 0.0127/(0.8 \times 2.3)$$
$$= 0{,}00690\ m^2$$
$$\text{Pipe radius} = \text{sqrt}\,(A/\pi)$$
$$= \text{sqrt}\,(0.00648/3.14159)$$
$$= 0.0468\ m$$

The pipe radius of 0.0468 m is not close enough to the original assumption of 0.05 m; therefore, recalculate using R = 0.047 m.

$$H_{o\text{–max}} = 0.424 – 0.1035 – 0.047 = 0.2735\ m$$
$$A = 0.00685\ m^2$$
$$\text{Pipe radius} = 0.0467\ m\ \text{(within acceptable range)}$$

Step 5—Compute the Flow Rate from the Pipe to Be Used for the Water Balance. The outlet flow rate through the underdrains (Q_U) needs to be constant to conduct a preliminary estimate of required water storage volume. Using the maximum flow rate out of the pipe for Q_U would overestimate the actual pipe discharge in all but extreme conditions. A representative percentage of the maximum flow rate should be used in the modeling for Q_U based on engineering judgment. In this example, a conservative value of Q_U is calculated using a differential head (Δh) equal to the pipe radius of 0.047 m.

$$Q_U = 0.8 \times (\pi \times 0.047^2) \times \text{sqrt}\,(2 \times 9.80665 \times 0.047)$$
$$= 0.8 \times (0.00694) \times (0.9601)$$
$$= 0.00533\ m^3/s$$
$$= 19.2\ m^3/h$$

Step 6—Calculate the Maximum Volume of Water to Be Stored in the Pavement Base (V_W). From Eq. (4-13),

$$V_W = P(A_P) + R(A_C) – I(T_S)A_I – Q(T_S)Z$$
$$V_W = (0.076 \times 4{,}000) + (0.07448 \times 8{,}000) – 0.0023 \times 24 \times 4{,}000 – 19.2 \times 24 \times 0.757$$

$$= 304 + 596 - 220.8 - 348.8$$

$$= 330.4 \text{ m}^3$$

Step 7—Calculate the Required Depth of the Base and/or Subbase. Calculate the hydraulic depth required using Eq. (4-14):

$$\text{Hydraulic depth} = \frac{V_w}{n * A_P}$$

$$= 330.4/(0.4 \times 4{,}000)$$

$$= 0.2065 \text{ m (8.12 in.)}$$

Step 8—Check That the Reduction in T_D in Step 2 Was Appropriate. Summarize the design details:

$Q_U = 19.2 \text{ m}^3/\text{h}$

$V_W = 330.4 \text{ m}^3$

Hydraulic depth = 0.2065 m

PE = 0.1035 m

$I = 0.0023$ m/h

The initial assumption was that it would take 6 h for the water level to drop to the bottom of the discharge point; that left 18 h for the system to completely drain without exceeding the maximum storage time of 24 h.

For the 6-h period, there would be both pipe flow and infiltration; the total discharge would be

$$6 \times (Q_U + I \times A_I) = 6 \times (19.2 + 0.0023 \times 4{,}000) = 6 \times 28.4 = 170.4 \text{ m}^3$$

The remaining 160 m^3 of the V_w would need to be below the pipe elevation for no pipe flow to occur.

The infiltration storage, which is the volume of water that can be stored below the elevation of the outlet structure, is based on the pipe elevation; the available infiltration storage is

$$\text{PE} \times A_I \times n = 0.1035 \times 4{,}000 \times 0.4 = 165.6 \text{ m}^3 \text{ (1,706 ft}^3\text{)}$$

Because the infiltration storage is approximately equal to the remainder of the V_W after 6 h, the original assumption was appropriate.

APPENDIX B-1

GUIDE CONSTRUCTION SPECIFICATION, PERMEABLE INTERLOCKING CONCRETE PAVEMENT, U. S. Version

Note: This guide specification for U.S. applications describes construction of permeable interlocking concrete pavers with jointing, bedding, base, and subbase aggregates. The joints between the pavers are typically filled with ASTM No. 8, 89, or 9 aggregates on a bedding layer of typically ASTM No. 8 stone. This 2-in.-thick layer is placed over an open-graded base (typically No. 57 aggregate) no greater than 4 in. thick. The base typically rests on a subbase (typically No. 2 aggregate or similar-sized material, such as No. 3 or 4 aggregate) whose thickness depends on water storage and traffic support requirements. In low-infiltration soils or installations with impermeable liners, some or all drainage is directed to an outlet via perforated drainpipes in the subbase. Although this guide specification does not cover excavation, liners, and drainpipes, notes are provided on these aspects.

The text must be edited to suit specific project requirements. It should be reviewed by a qualified civil or geotechnical engineer familiar with the site conditions. Edit terms in this specification as necessary to identify the design professional in the general conditions of the contract.

PART 1 GENERAL

1.1 SUMMARY

A. Section includes
  1. Permeable interlocking concrete pavers.
  2. Open-graded aggregate bedding material.
  3. Open-graded base aggregate.
  4. Open-graded subbase aggregate.
  5. Jointing aggregate.
  6. Edge restraints.
  7. [Geotextiles].

B. Related sections
  1. Section [_______]: Curbs.
  2. Section [_______]: [Stabilized] aggregate base.
  3. Section [_______]: [PVC] Drainage pipes.

4. Section [_______]: Impermeable liner.
5. Section [_______]: Edge restraints.
6. Section [_______]: Drainage pipes and appurtenances.
7. Section [_______]: Earthworks, excavation, and soil compaction.

1.2 Consensus Standards and Other Referenced Documents

A. American Society for Testing and Materials (ASTM)

1. **ASTM C131**, *Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine*, 2014a.
2. **ASTM C136**, *Standard Test Method for Sieve Analysis for Fine and Coarse Aggregates*, 2014b.
3. **ASTM C140**, *Standard Test Methods for Sampling and Testing Concrete Masonry Units and Related Units*, 2017c.
4. **ASTM D448**, *Standard Classification for Sizes of Aggregate for Road and Bridge Construction*, 2012a.
5. **ASTM C936**, *Standard Specification for Solid Concrete Interlocking Paving Units*, 2016c.
6. **ASTM C979**, *Specification for Pigments for Integrally Colored Concrete*, 2016d.
7. **ASTM C1645**, *Standard Test Method for Freeze–Thaw and De-icing Salt Durability of Solid Concrete Interlocking Paving Units*, 2016e.
8. **ASTM C1781**, *Standard Test Method for Surface Infiltration Rate of Permeable Unit Pavement Systems*, 2015.
9. **ASTM D698**, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3))*, 2012b.
10. **ASTM D3385**, *Standard Test Method for Infiltration Rate of Soils in Field using Double-Ring Infiltrometer*, 2009.
11. **ASTM E2835**, *Standard Test Method for Measuring Deflections using a Portable Impulse Plate Load Test Device*, 2011b.

B. Interlocking Concrete Pavement Institute (ICPI)

1. *Permeable Interlocking Concrete Pavement Specialist Course*, 2017b.
2. *Permeable Pavement Design Manual*, 2017c.
3. *Permeable Design Pro* software for hydrologic and structural design, 2010.

1.3 SUBMITTALS

A. In accordance with conditions of the contract and Division 1 Submittal Procedures Section.

B. The paver manufacturer’s or installation subcontractor’s drawings and details: Indicate perimeter conditions, junction with other materials, expansion and control joints, paver [layout,] [patterns,] [color arrangement,] installation [and setting] details. Indicate layout, pattern, and relationship of paving joints to fixtures, and project formed details.

C. Minimum 3-lb samples of subbase, base, and bedding aggregate materials.

D. Sieve analysis of aggregates for subbase, base, and bedding materials, per ASTM C136.

E. Project-specific or producer or manufacturer source test results for void ratio and bulk density of the base and subbase aggregates.

F. Soils report indicating density test reports, classification, and infiltration rate measured on site under compacted conditions, and suitability for the intended project.

G. Erosion and sediment control plan.

H. [Stormwater management [quality][quantity] calculations; structural analysis for vehicular applications] or [specify] design methods and models per ASCE [XX], ICPI *Permeable Interlocking Concrete Pavement Design Manual*, or *Permeable Design Pro* software program.

Commented [GM1]: Add when available

I. Permeable concrete pavers:

   1. Paver manufacturer’s catalog sheets with product specifications.
   2. [Four] representative full-size samples of each paver type, thickness, color, and finish. Submit samples indicating the range of color expected in the finished installation.
   3. Accepted samples become the standard of acceptance for the work of this section.
   4. Laboratory test reports certifying compliance of the concrete pavers with ASTM C936.
   5. Manufacturer’s certification of concrete pavers by ICPI as having met applicable ASTM standards.
   6. Manufacturers’ material safety data sheets for the safe handling of the specified paving materials and other products specified herein.
   7. Paver manufacturer’s written quality control procedures, including representative samples of production record keeping that ensure conformance of paving products to the product specifications.

J. Paver Installation Subcontractor:

   1. Demonstrate that job supervisors on the project have a current certificate from the ICPI’s Concrete Paver Installer Certification program and a record of completion from the PICP Installer Course.

2. Job references from projects of a similar size and complexity. Provide name and contact information for owner, client, and general contractor.
3. Written method statement and quality control plan that describe material staging and flow, paving direction, and installation procedures (e.g., layer stitching, as applicable), including representative reporting forms that ensure conformance to the project specifications.

1.4 QUALITY ASSURANCE

A. Paver Installation Subcontractor Qualifications:
   1. Use an installer who has successfully completed concrete paver installation similar in design, material, and extent indicated on this project.
   2. Use an installer with job supervisors who hold a record of completion from the Interlocking Concrete Pavement Institute’s PICP Installer Technician course.

B. Regulatory Requirements and Approvals: [Specify applicable licensing, bonding, or other requirements of regulatory agencies.].

C. Review the manufacturers’ quality control plan, paver installation subcontractor’s method statement and quality control plan at a preconstruction meeting of representatives from the manufacturer, paver installation subcontractor, general contractor, engineer, and/or owner’s representative.

D. Mock-ups:
   1. Install a 10 ft × 10 ft paver area.

Note: Mechanized installations may require a larger mock-up area. Consult with the paver installation contractor on the size of the mock-up.

   2. Use this area to determine surcharge of the bedding layer, joint sizes and lines, laying pattern, color, and texture of the job.
   3. This area will be used as the standard by which the work will be judged.
   4. Subject to acceptance by the owner, mock-up may be retained as part of the finished work.
   5. If the mock-up is not retained, remove and properly dispose of the mock-up.

1.5 DELIVERY, STORAGE, AND HANDLING

A. General: Comply with Division 1 product requirement section.

B. Comply with manufacturer’s ordering instructions and lead-time requirements to avoid construction delays.

C. Delivery: Deliver materials in manufacturer’s original, unopened, undamaged container packaging with identification tags intact on each paver bundle.
  1. Coordinate delivery and paving schedule to minimize interference with normal use of buildings adjacent to paving.
  2. Deliver concrete pavers to the site in steel-banded, plastic-banded, or plastic-wrapped cubes capable of transfer by forklift or clamp lift.
  3. Unload pavers at job site in such a manner that no damage occurs to the product or existing construction.

D. Storage and Protection: Store materials in protected area such that they are kept free from mud, dirt, and other foreign materials.

1.6 ENVIRONMENTAL REQUIREMENTS

A. Do not install in heavy rain.
B. Do not install frozen aggregates.
C. Do not install aggregates on frozen soil subgrade.

1.7 MAINTENANCE

A. Extra materials: Provide [Specify area] [Specify percentage] additional material for use by owner for maintenance and repair.
B. Pavers shall be from the same production run as installed materials.

PART 2 PRODUCTS

Note: Some projects may include permeable and solid interlocking concrete pavements. Specify each product as required.

2.1 PAVING UNITS

A. Manufacturer: [Specify ICPI member manufacturer name].
  1. Contact: [Specify ICPI member manufacturer contact information].

B. Permeable Interlocking Concrete Paver Units:
  1. Paver Type: [Specify name of product group, family, and/or series].
    a. Material Standard: Comply with ASTM C936. Use –15°C as the lowest temperature for freeze-thaw durability testing while test specimens are immersed in a 3% saline solution, per ASTM C1645.
    b. Color [and finish]: [Specify color.] [Specify finish].

c. Color Pigment Material Standard: Comply with ASTM C979.

Note: Concrete pavers may have spacer bars on each unit. Spacer bars are recommended for mechanically installed pavers. Manually installed pavers may be installed with or without spacer bars. Verify with manufacturers that overall dimensions do not include spacer bars.

d. Size: [Specify.] in. long × [Specify.] in. wide × [Specify.] in. thick.

2.2 PRODUCT SUBSTITUTIONS

A. Substitutions: Permitted for gradations for crushed stone jointing material and base and subbase materials. Base and subbase materials shall have a minimum 0.32 porosity. All substitutions shall be approved in writing by the project engineer.

2.3 JOINTING, BEDDING, BASE, AND SUBBASE AGGREGATES

A. Crushed stone with 90% fractured faces, LA abrasion < 40, per ASTM C131.

B. Do not use rounded river gravel or recycled concrete aggregates for vehicular applications.

C. All stone materials shall be washed with less than 2% passing the No. 200 sieve.

D. Joint and/or opening filler, bedding, base, and subbase conforming to ASTM D448 gradation, as shown in Tables B-1-1, B-1-2, and B-1-3, with less than 2% passing the No. 200 sieve.

Note: No. 89 or No. 9 aggregates may be used to fill pavers with narrow joints and for bedding aggregate if the choke criteria with the underlying base gradation is satisfied.

Table B-1-1. ASTM No. 8 Grading Requirements for Jointing and Bedding Aggregates

Sieve Size	*Percent Passing*
1/2 in.	100
3/8 in.	85 to 100
No. 4	10 to 30
No. 8	0 to 10
No. 16	0 to 5

Table B-1-2. ASTM No. 57 Grading Requirements for Base Aggregates

Sieve Size	*Percent Passing*
1 1/2 in.	100
1 in.	95 to 100
1/2 in.	25 to 60
No. 4	0 to 10
No. 8	0 to 5

Note: ASTM No. 3 or No. 4 stone may be used as subbase material if ASTM No. 2 stone is unavailable.

Table B-1-3. ASTM No. 2 Grading Requirements for Subbase Aggregate

Sieve Size	*Percent Passing*
3 in.	100
2 1/2 in.	90 to 100
2 in.	35 to 70
1 1/2 in.	0 to 15
3/4 in.	0 to 5

2.4 ACCESSORIES

A. Provide accessory materials as follows:

Note: Curbs are typically cast-in-place concrete or precast set in concrete haunches. Concrete curbs may be specified in another section. Do not use plastic edging with steel spikes to restrain the paving units for vehicular applications.

1. Edge Restraints
    - a. Manufacturer: [Specify manufacturer].
    - b. Material: [Precast concrete] [Cut stone] [Concrete].
    - c. Material Standard: [Specify material standard].

Note: See ICPI publication, Permeable Interlocking Concrete Pavements, *for guidance on geotextile selection. Geotextile use is a designer option.*

2. Geotextile:
    - a. Material Type and Description: [Specify material type and description].
    - b. Material Standard: [Specify material standard].
    - c. Manufacturer: [Acceptable to interlocking concrete paver manufacturer].

PART 3 EXECUTION

3.1 ACCEPTABLE INSTALLERS

A. [Specify acceptable paver installation subcontractors.].

3.2 EXAMINATION

Note: The elevations and surface tolerance of the soil subgrade determine the final surface elevations of concrete pavers. The paver installation contractor cannot correct deficiencies of excavation and grading of the soil subgrade with additional bedding materials. Therefore, the surface elevations of the soil subgrade should be checked and accepted by the general contractor or designated party, with written certification presented to the paver installation subcontractor before starting work.

A. Acceptance of Site Verification of Conditions:

1. General contractor shall inspect, accept, and certify in writing to the paver installation subcontractor that site conditions meet specifications for the following items before installation of interlocking concrete pavers.

Note: Compaction of the soil subgrade is optional and should be determined by the project engineer. If the soil subgrade requires compaction, it should be compacted to a minimum of 95% standard Proctor density, per ASTM D698. Compacted soil density and moisture should be checked in the field with a nuclear density gauge or other test methods for compliance to specifications. Stabilization of the soil and/or base material may be necessary with weak or continually saturated soils, or when subject to high wheel loads. Compaction reduces the permeability of soils. If soil compaction is necessary, the infiltration rate should be estimated per ASTM D3385 for hydrologic design after compacting the test area(s) and measuring density. Reduced infiltration may require drainpipes within the open-graded subbase to conform to local storm drainage requirements.

   a. Verify that subgrade preparation, compacted density, and elevations conform to specified requirements.
   b. Provide written density test results for soil subgrade to the owner, general contractor, and paver installation subcontractor.
   c. Verify location, type, and elevations of edge restraints, [concrete collars around] utility structures, and drainage pipes and inlets.

2. Do not proceed with installation of bedding and interlocking concrete pavers until subgrade soil conditions are corrected by the general contractor or designated subcontractor.

3.3 PREPARATION

A. Verify that the soil subgrade is free from standing water.

B. Stockpile joint or opening filler, base, and subbase materials such that they are free from standing water; uniformly graded; free of any organic material, sediment, or debris; and ready for placement.

C. Edge Restraint Preparation:

   1. Install edge restraints per the drawings [at the indicated elevations].

3.4 INSTALLATION

Note: The minimum slope of the soil subgrade is typically 0.5%. Actual slope of soil subgrade depends on the drainage design and exfiltration type. All drainpipes, observation wells, overflow pipes, and (if applicable) geotextiles, berms, baffles, and impermeable liners should be in place per the drawings *before or during placement of the subbase and base, depending on their location. Care must be taken not to damage drainpipes during compaction and paving. Base and subbase thicknesses and drainage should be determined using ICPI's* Permeable Interlocking Concrete Pavements Manual *and* Permeable Design Pro *software.*

A. General

1. Any excess thickness of soil applied over the excavated soil subgrade to trap sediment from adjacent construction activities shall be removed before application of the [geotextile] and subbase materials.
2. Keep area where pavement is to be constructed free from sediment during entire job. [Geotextiles] Base and bedding materials contaminated with sediment shall be removed and replaced with clean materials.
3. Do not damage drainpipes, overflow pipes, observation wells, or any inlets and other drainage appurtenances during installation. Report any damage immediately to the project engineer.

B. Geotextiles

1. Place on [bottom and] sides of soil subgrade. Secure in place to prevent wrinkling from vehicle tires and tracks.
2. Overlap a minimum of 12 in. or in the direction of drainage.

C. Open-Graded Subbase and Base

Note: Compaction of areas or sites that cannot accommodate a roller vibratory compactor may use a minimum 13,500-lbf vibratory plate compactor with a compaction indicator. At least two passes should be made over each lift of the subbase and base aggregates.

1. Moisten, spread, and compact the No. 2 subbase in maximum 8-in.-thick lifts [without wrinkling or folding the geotextile. Place subbase to protect geotextile from wrinkling under equipment tires and tracks.]
2. For each lift, make at least two passes in the vibratory mode, then at least two in the static mode with a minimum 10-t vibratory roller until there is no visible movement of the No. 2 stone. Do not crush aggregate with the roller.
3. Use a minimum 13,500-lbf plate compactor with a compaction indicator to compact areas that cannot be reached by the vibratory roller. Do not crush the aggregate with the plate compactor.
4. The surface tolerance of the compacted No. 2 subbase shall be ± 2 1/2 in. over a 10 ft straightedge.

5. Moisten, spread, and compact the No. 57 base layer in one 4 in.-thick lift. On this layer, make at least two passes in the vibratory mode, then at least two in the static mode with a minimum 10-t vibratory roller until there is no visible movement of the No. 57 stone. Do not crush aggregate with the roller.
6. The surface tolerance of the compacted No. 57 base should not deviate more than ± 1 in. over a 10 ft straightedge.

Note: At the option of the designer, this supplemental test method bracketed in item C7 describing the use of a lightweight deflectometer (LWD) can be used for in situ deflection testing of the compacted. open-graded aggregate subbase layer (typically ASTM No. 2, 3, or 4 stone) and the compacted base layer (typically ASTM No. 57 stone). This test method is appropriate for pavements subject to consistent vehicular traffic, such as parking lots, alleys, and roads and can be used on pedestrian and residential driveway projects to help minimize settlement. The LWD test method should comply with ASTM E2835. This test protocol is not needed for pedestrian areas and residential driveways.

7. LWD for compacted subbase and base aggregate deflection testing
   a. Test a minimum of every 500 ft^2 of compacted subbase and base area. In addition, test areas next to other pavements, curbs, buildings, and protrusions.
   b. Do not test aggregate over saturated soil subgrades.
   c. The maximum average of three deflections deemed acceptable shall be 0.02 in.
8. Report
   a. The report shall include the following:
      1. Project description.
      2. Aggregate type and layer thicknesses.
      3. Aggregate characteristic properties: gradation, porosity, and bulk density.
      4. Compaction equipment type and weight.
      5. Static and/or vibratory compaction.
      6. Number of passes of the compaction equipment.
      7. Sketch of test area and numbered LWD test locations on the compacted subbase and base.
      8. Average of three LWD deflections for each location in fractions of an inch.]

D. Bedding Layer

1. Moisten, spread, and screed the No. 8 stone bedding material.
2. Fill voids left by removed screed rails with No. 8 stone and smooth to conform to adjacent screeded bedding material.

3. The surface tolerance of the screeded No. 8 bedding layer shall be ±3/8 in. over a 10 ft straightedge.
4. Do not subject screeded bedding material to any pedestrian or vehicular traffic before paving unit installation begins.

E. Permeable Interlocking Concrete Pavers and Jointing Aggregates

1. Lay the paving units in the pattern(s) and joint widths shown on the drawings. Maintain straight pattern lines.
2. Fill gaps at the edges of the paved area with cut units. Cut pavers subject to tire traffic shall be no smaller than one third of a whole unit. Cut pavers placed in other areas shall no less than 2 in. long.
3. Cut pavers with a masonry saw and place them at the edges.

Note: Some paver joint widths may be narrow and may not accept most of the No. 8 stone. Use joint material that will fill joints such as washed ASTM No. 89 or No. 9 stone.

4. Compact within 6 ft of the unrestrained edges of the paving units using a low-amplitude, 75–90 Hz plate compactor capable of at least 5,000 lbf. Make at least two passes in perpendicular directions with the plate compactor.
5. Apply jointing stone to the paver surface, sweeping it across and filling the paver joints.
6. Remove excess aggregate on the surface by sweeping the pavers clean.
7. Compact and seat the pavers into the bedding material using a low-amplitude, 75–90 Hz plate compactor capable of at least 5,000 lbf. Make at least two passes in perpendicular directions with the plate compactor.
8. Apply additional aggregate to the openings and joints if needed, filling them completely.
9. Remove excess aggregate by sweeping.
10. All pavers within 6 ft of the laying face must be left fully compacted at the completion of each day.
11. The final surface tolerance of compacted pavers shall not deviate more than ±3/8 in. under a 10 ft straightedge.
12. The surface elevation of pavers shall be 1/8 to 1/4 in. above adjacent drainage inlets, concrete collars, or channels.

3.5 FIELD QUALITY CONTROL

A. After sweeping the surface clean, check final elevations for conformance to the drawings.

B. Lippage: No greater than 1/8 in. difference in height between adjacent pavers.

Note: The surface of the pavers may be 1/8 to 1/4 in. above the final elevations after compaction. This height helps compensate for possible minor settling normal to pavements.

C. The surface elevation of pavers shall be 1/8 to 1/4 in. above adjacent drainage inlets, concrete collars, or channels.

D. Bond lines for paver courses: ±1/2 in. over a 50 ft taut string line.

E. Verify the surface infiltration at a minimum of 100 in./h using test method ASTM C1781.

3.6 PROTECTION

A. After work in this section is complete, the general contractor shall be responsible for protecting work from sediment deposition and damage caused by subsequent construction activity on the site.

B. The PICP installation contractor shall return to the site after six months from the completion of the work and provide the following as required: filling paver joints with stones, replacing broken or cracked pavers, and releveling settled pavers to initial elevations. Any additional work shall be considered part of the original bid price, with no additional compensation.

APPENDIX B-2

GUIDE CONSTRUCTION SPECIFICATION, PERMEABLE INTERLOCKING CONCRETE PAVEMENT, CANADIAN VERSION

Note: This guide specification for Canadian applications describes construction of permeable interlocking concrete pavers with jointing, bedding, base, and subbase aggregates. The joints between the pavers are typically filled with CSA A23.1 Group II, 5-2.5 mm nominal size aggregate (or similar) and are typically placed on a permeable, open-graded crushed stone bedding layer (typically CSA A23.1 Group II, 10-5 mm nominal size aggregate). This 50 mm-thick bedding layer is placed over an open-graded base (typically CSA A23.1 Group II, 28-14 mm nominal size aggregate) that is 100 mm thick. This base rests on a subbase (typically CSA A23.1 Group II, 80-40 mm nominal size aggregate) whose thickness depends on water storage and traffic support requirements. In low-infiltration soils or installations with impermeable liners, some or all drainage is directed to an outlet via perforated drainpipes in the subbase. Although this guide specification does not cover excavation, liners, and drainpipes, notes are provided on these aspects.

The text must be edited to suit specific project requirements. It should be reviewed by a qualified civil or geotechnical engineer familiar with the site conditions. Edit terms in this specification as necessary to identify the design professional in the general conditions of the contract.

PART 1 GENERAL

1.1 SUMMARY

A. Section includes
  1. Permeable interlocking concrete pavers
  2. Coarse aggregate bedding material
  3. Coarse base aggregate
  4. Open-graded subbase aggregate
  5. Jointing aggregate
  6. Edge restraints, and
  7. [Geotextiles].

B. Related Sections

1. Section [_______]: Curbs.
2. Section [_______]: [Stabilized] aggregate base.
3. Section [_______]: [PVC] Drainage pipes.
4. Section [_______]: Impermeable liner.
5. Section [_______]: Edge restraints.
6. Section [_______]: Drainage pipes and appurtenances.
7. Section [_______]: Earthworks, excavation, and soil compaction.

1.2 Consensus Standards and Other Referenced Documents

A. American Society for Testing and Materials (ASTM)

1. **ASTM C131**, *Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine*, 2014a.
2. **ASTM C1781**, *Standard Test Method for Surface Infiltration Rate of Permeable Unit Pavement Systems*, 2015a.
3. **ASTM D698**, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort [(12,400 ft-lbf/ft^3 (600 kN-m/m^3)]*, 2012d.
4. **ASTM D3385**, *Standard Test Method for Infiltration Rate of Soils in Field Using Double-Ring Infiltrometer*, 2009c.
5. **ASTM E2835**, *Standard Test Method for Measuring Deflections using a Portable Impulse Plate Load*, 2015c.

B. Canadian Standards Association (CSA)

1. **CSA A23.1/A23.2**, *Concrete Materials and Methods of Concrete Construction/ Test Methods and Standard Practices for Concrete*, 2014a.
2. **CSA A23.2A** (A23.2-14), *Sieve Analysis of Fine and Coarse Aggregates*, 2014c.
3. **CSA A23.2-10A**, *Density of Aggregate. Concrete Materials and Methods of Concrete Construction* (2014d).
4. **CSA A23.2-16A**, *Resistance to Degradation of Small-size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine (for Aggregate $\leq$ 40 mm)*, 2014d.
5. **CSA A23.2-17A**, *Resistance to Degradation of Large-size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine (for Aggregate > 40 mm)*, 2012.
6. **CSA A231.2**, *Precast Concrete Pavers*, 2014b.

D. Interlocking Concrete Pavement Institute (ICPI)

1. *Permeable Interlocking Concrete Pavement Specialist* Course, 2017b.

2. *Permeable Interlocking Concrete Pavement Design Manual*, 2017c.
3. *Permeable Design Pro* software for hydrologic and structural design, 2010.

1.3 SUBMITTALS

A. In accordance with conditions of the contract and Division 1 Submittal Procedures Section.

B. The paver manufacturer's or installation subcontractor's drawings and details: Indicate perimeter conditions, junction with other materials, expansion and control joints, paver [layout,] [patterns,] [color arrangement,] installation [and setting] details. Indicate layout, pattern, and relationship of paving joints to fixtures, and project formed details.

C. Minimum 2 kg samples of subbase, base, and bedding aggregate materials.

D. Sieve analysis of aggregates for subbase, base, bedding, and jointing materials, per CSA A23.2A.

E. Project-specific or producer or manufacturer source test results for porosity and bulk density of the base and subbase aggregates, per CSA A23.2-10A.

F. Soils report indicating density test reports, classification, and infiltration rate measured on site under compacted conditions, and suitability for the intended project.

G. Erosion and sediment control plan.

H. [Stormwater management [quality][quantity] calculations; structural analysis for vehicular applications] or [specify] design methods and models per ASCE [XX-xx], ICPI's *Permeable Interlocking Concrete Pavement Manual*, or *Permeable Design Pro* software program.

Commented [GM1]: Add when available

I. Permeable concrete pavers:

1. Paver manufacturer's catalog sheets with product specifications.
2. [Four] representative full-size samples of each paver type, thickness, color, and finish. Submit samples indicating the range of color expected in the finished installation.
3. Accepted samples become the standard of acceptance for the work of this section.
4. Laboratory test reports certifying compliance of the concrete pavers with CSA A231.2.
5. Manufacturer's certification of concrete pavers by ICPI as having met applicable ASTM standards.
6. Manufacturers' material safety data sheets for the safe handling of the specified paving materials and other products specified herein.
7. Paver manufacturer's written quality control procedures, including representative samples of production record keeping that ensure conformance of paving products to the product specifications.

J. Paver Installation Subcontractor:

1. Demonstrate that job supervisors on the project have a current certificate from the Interlocking Concrete Pavement Institute's Concrete Paver Installer Certification program and a record of completion from the PICP Installer Course.
2. Job references from projects of a similar size and complexity. Provide name and contact information for owner, client, and general contractor.
3. Written method statement and quality control plan that describe material staging and flow, paving direction, and installation procedures (e.g., layer stitching, as applicable), including representative reporting forms that ensure conformance to the project specifications.

1.4 QUALITY ASSURANCE

A. Paver Installation Subcontractor Qualifications:

1. Use an installer who has successfully completed concrete paver installation similar in design, material, and extent indicated on this project.
2. Use an installer with job supervisors who hold a record of completion from the Interlocking Concrete Pavement Institute's PICP Installer Technician course.

B. Regulatory Requirements and Approvals: [Specify applicable licensing, bonding, or other requirements of regulatory agencies.].

C. Review the manufacturers' quality control plan, paver installation subcontractor's method statement and quality control plan at a preconstruction meeting of representatives from the manufacturer, paver installation subcontractor, general contractor, engineer, and/or owner's representative.

D. Mock-Ups:

1. Install a 3 m × 3 m paver area.

Note: Mechanized installations may require a larger mock-up area. Consult with the paver installation contractor on the size of the mock-up.

2. Use this area to determine surcharge of the bedding layer, joint sizes and lines, laying pattern, color, and texture of the job.
3. This area will be used as the standard by which the work will be judged.
4. Subject to acceptance by the owner, mock-up may be retained as part of the finished work.
5. If the mock-up is not retained, remove and properly dispose of the mock-up.

1.5 DELIVERY, STORAGE, AND HANDLING

A. General: Comply with Division 1 product requirement section.

B. Comply with manufacturer's ordering instructions and lead-time requirements to avoid construction delays.

C. Delivery: Deliver materials in manufacturer's original, unopened, undamaged container packaging with identification tags intact on each paver bundle.

   1. Coordinate delivery and paving schedule to minimize interference with normal use of buildings adjacent to paving.
   2. Deliver concrete pavers to the site in steel-banded, plastic-banded, or plastic-wrapped cubes capable of transfer by forklift or clamp lift.
   3. Unload pavers at job site in such a manner that no damage occurs to the product or existing construction.

D. Storage and Protection: Store materials in protected area such that they are kept free from mud, dirt, and other foreign materials.

1.6 ENVIRONMENTAL REQUIREMENTS

A. Do not install in heavy rain.

B. Do not install frozen aggregates.

C. Do not install aggregates on frozen soil subgrade.

1.7 MAINTENANCE

A. Extra materials: Provide [Specify area] [Specify percentage] additional material for use by owner for maintenance and repair.

B. Pavers shall be from the same production run as installed materials.

PART 2 PRODUCTS

Note: Some projects may include permeable and solid interlocking concrete pavements. Specify each product as required.

2.1 PAVING UNITS

A. Manufacturer: [Specify ICPI member manufacturer name.].

   1. Contact: [Specify ICPI member manufacturer contact information.].

B. Permeable Interlocking Concrete Paver Units:

1. Paver Type: [Specify name of product group, family, and/or series].
    a. Material Standard: Comply with CSA A231.2 compressive strength and deicer durability requirements. Color [and finish]: [Specify color.] [Specify finish].

Note: Concrete pavers may have spacer bars on each unit. Spacer bars are recommended for mechanically installed pavers. Manually installed pavers may be installed with or without spacer bars. Verify with manufacturers that overall dimensions do not include spacer bars.

    d. Size: [Specify] mm long × [Specify] mm wide × [Specify] mm thick.

2.2 PRODUCT SUBSTITUTIONS

A. Substitutions: Permitted for gradations for crushed stone jointing material and base and subbase materials. Base and subbase materials shall have a minimum 0.32 porosity. All substitutions shall be approved in writing by the project engineer.

2.3 CRUSHED STONE FILLER, BEDDING, BASE, AND SUBBASE

A. Crushed stone with 90% fractured faces, LA abrasion < 40, per CSA A23.2-16A or A23.2-17A, as applicable to the largest aggregate size of each material gradation.

B. Do not use rounded river gravel or recycled concrete aggregates for vehicular applications.

C. All stone materials shall be washed with less than 2% passing the 0.075-mm sieve.

D. Joint and/or opening filler, bedding, base, and subbase conforming to CSA A23.1 Group II, grading requirements for coarse aggregates, as shown in Tables B-2-1, B-2-2, and B-2-3.

Note: Group II 5-2.5 mm aggregate may be used to fill paver joints and for bedding aggregate if the choke criteria with the underlying base gradation is satisfied. Confirm recommended gradations from the concrete paver supplier.

Table B-2-1. 10-2.5 mm Aggregate Grading Requirements for the Bedding Layer

Sieve Size	*Percent Passing*
14 mm	100
10 mm	85 to 100
5 mm	10 to 30
2.5 mm	0 to 10
1.25 mm	0 to 5

Table B-2-2. 28-5 mm Aggregate Grading Requirements for the Base

Sieve Size	*Percent Passing*
40 mm	100
28 mm	95 to 100

14 mm	30 to 65
5 mm	0 to 10
2.5 mm	0 to 5

Note: 56-28 mm size aggregate may be used as subbase material.

Table B-2-3. 80-40 mm Aggregate Grading Requirements for the Subbase

Sieve Size	*Percent Passing*
112 mm	100
80 mm	90 to 100
56 mm	25 to 60
40 mm	0 to 15
20 mm	0 to 5

2.4 ACCESSORIES

A. Provide accessory materials as follows:

Note: Curbs are typically cast-in-place concrete or precast set in concrete haunches. Concrete curbs may be specified in another section. Do not use plastic edging with steel spikes to restrain the paving units for vehicular applications.

1. Edge Restraints
    a. Manufacturer: [Specify manufacturer.].
    b. Material: [Precast concrete] [Cut stone] [Concrete].
    c. Material Standard: [Specify material standard.].

Note: See ICPI publication, Permeable Interlocking Concrete Pavements, *for guidance on geotextile selection. Geotextile use is a designer option.*

2. Geotextile:
    a. Material Type and Description: [Specify material type and description].
    b. Material Standard: [Specify material standard].
    c. Manufacturer: [Acceptable to interlocking concrete paver manufacturer].

PART 3 EXECUTION

3.1 ACCEPTABLE INSTALLERS

A. [Specify acceptable paver installation subcontractors.].

3.2 EXAMINATION

Note: The elevations and surface tolerance of the soil subgrade determine the final surface elevations of concrete pavers. The paver installation contractor cannot correct deficiencies of excavation and grading *of the soil subgrade with additional bedding materials. Therefore, the surface elevations of the soil subgrade should be checked and accepted by the general contractor or designated party, with written certification presented to the paver installation subcontractor before starting work.*

A. Acceptance of Site Verification of Conditions:

1. General contractor shall inspect, accept, and certify in writing to the paver installation subcontractor that site conditions meet specifications for the following items before installation of interlocking concrete pavers.

Note: Compaction of the soil subgrade is optional and should be determined by the project engineer. If the soil subgrade requires compaction, it should be compacted to a minimum of 95% standard Proctor density, per ASTM D698. Compacted soil density and moisture should be checked in the field with a nuclear density gauge or other test methods for compliance to specifications. Stabilization of the soil and/or base material may be necessary with weak or continually saturated soils, or when subject to high wheel loads. Compaction reduces the permeability of soils. If soil compaction is necessary, the infiltration rate should be estimated per ASTM D3385 for hydrologic design after compacting the test area(s) and measuring density. Reduced infiltration may require drainpipes within the open-graded subbase to conform to local storm drainage requirements.

   a. Verify that subgrade preparation, compacted density, and elevations conform to specified requirements.
   b. Provide written density test results for soil subgrade to the owner, general contractor, and paver installation subcontractor.
   c. Verify location, type, and elevations of edge restraints, [concrete collars around] utility structures, and drainage pipes and inlets.

2. Do not proceed with installation of bedding and interlocking concrete pavers until subgrade soil conditions are corrected by the general contractor or designated subcontractor.

3.3 PREPARATION

A. Verify that the soil subgrade is free from standing water.

B. Stockpile joint or opening filler, base, and subbase materials such that they are free from standing water; uniformly graded; free of any organic material, sediment, or debris; and ready for placement.

C. Edge Restraint Preparation:

1. Install edge restraints per the drawings [at the indicated elevations].

3.4 INSTALLATION

Note: The minimum slope of the soil subgrade is typically 0.5%. Actual slope of soil subgrade depends on the drainage design and exfiltration type. All drainpipes, observation wells, overflow pipes, and (if applicable) geotextiles, berms, baffles, and impermeable liners should be in place per the drawings before or during placement of the subbase and base, depending on their location. Care must be taken not to damage drainpipes during compaction and paving. Base and subbase thicknesses and drainage should be determined using ICPI's Permeable Interlocking Concrete Pavements Manual *and* Permeable Design Pro *software.*

A. General
   1. Any excess thickness of soil applied over the excavated soil subgrade to trap sediment from adjacent construction activities shall be removed before application of the [geotextile] and subbase materials.
   2. Keep area where pavement is to be constructed free from sediment during entire job. [Geotextiles] Base and bedding materials contaminated with sediment shall be removed and replaced with clean materials.
   3. Do not damage drainpipes, overflow pipes, observation wells, or any inlets and other drainage appurtenances during installation. Report any damage immediately to the project engineer.

B. Geotextiles
   1. Place on [bottom and] sides of soil subgrade. Secure in place to prevent wrinkling from vehicle tires and tracks.
   2. Overlap a minimum of 0.3 m in the direction of drainage.

C. Open-Graded Subbase and Base

Note: Compaction of areas or sites that cannot accommodate a roller vibratory compactor may use a minimum 60 kN vibratory plate compactor with a compaction indicator. At least two passes should be made over each lift of the subbase and base aggregates.

   1. Moisten, spread, and compact the 80-40 mm subbase aggregate in maximum 200 mm-thick lifts [without wrinkling or folding the geotextile. Place subbase to protect geotextile from wrinkling under equipment tires and tracks.]
   2. For each lift, make at least two passes in the vibratory mode, then at least two in the static mode with a minimum 9 metric ton vibratory roller until there is no visible movement of the 80-40 mm aggregate. Do not crush aggregate with the roller.

3. Use a minimum 60 kN plate compactor with a compaction indicator to compact areas that cannot be reached by the vibratory roller. Do not crush the aggregate with the plate compactor.
4. The surface tolerance of the compacted 80-40 mm aggregate shall be ±65 mm over a 3 m straightedge.
5. Moisten, spread, and compact the 28-5 mm aggregate base layer in one 100 mm-thick lift. On this layer, make at least two passes in the vibratory mode, then at least two in the static mode with a minimum 9 metric ton vibratory roller until there is no visible movement of the aggregate. Do not crush aggregate with the roller.
6. The surface tolerance of the compacted 28-5 mm aggregate base shall be ±25 mm over a 3-m straightedge.

Note: At the option of the designer, this supplemental test method bracketed in item C7 describing the use of a lightweight deflectometer (LWD) can be used for in situ deflection testing of the compacted, open-graded aggregate subbase layer and the compacted base layer. This test method can assist contractors in reaching adequate job site compaction and offer an additional level of confidence for the project owner and designer. This test method is appropriate for pavements subject to consistent vehicular traffic, such as parking lots, alleys, and roads and can be used on pedestrian and residential driveway projects to help minimize settlement. The LWD test method should comply with ASTM E2835.

7. LWD for compacted subbase and base aggregate deflection testing
    a. Test a minimum of every 50 m^2 of compacted subbase and base area. In addition, test areas next to other pavements, curbs, buildings, and protrusions.
    b. Do not test aggregates over saturated soil subgrades.
    c. The maximum average of three deflections deemed acceptable shall be 0.5 mm.
8. Report
    a. The report shall include the following:
        1. Project description.
        2. Aggregate type and layer thicknesses.
        3. Aggregate characteristic properties: gradation, porosity, and bulk density.
        4. Compaction equipment type and weight.
        5. Static and/or vibratory compaction.
        6. Number of passes of the compaction equipment.
        7. Sketch of test area and numbered LWD test locations on the compacted subbase and base.
        8. Average of three LWD deflections for each location in millimeters.

D. Bedding Layer

1. Moisten, spread, and screed the 10-2.5 mm aggregate.
2. Fill voids left by removed screed rails with 10-5 mm aggregate and smooth to conform to adjacent screeded bedding material.
3. The surface tolerance of the screeded 10-2.5 mm aggregate bedding layer shall be ±10 mm over a 3 m straightedge.
4. Do not subject screeded bedding material to any pedestrian or vehicular traffic before paving unit installation begins.

E. Permeable Interlocking Concrete Pavers and Joint or Opening Fill Material

1. Lay the paving units in the pattern(s) and joint widths shown on the drawings. Maintain straight pattern lines.
2. Fill gaps at the edges of the paved area with cut units. Cut pavers subject to tire traffic shall be no smaller than one third of a whole unit. Cut pavers placed in other areas shall no less than 50 mm long.
3. Cut pavers with a masonry saw and place them at the edges. Compact within 2 m of the unrestrained edges of the paving units using a low-amplitude, 75–90 Hz plate compactor capable of at least 22 kN.

Note: Some paver joint widths may be narrow and may not accept most of the No. 8 stone. Use joint material that will fill joints such as washed ASTM No. 89 or No. 9 stone.

4. Make at least two passes in perpendicular directions with the plate compactor.
5. Apply jointing stone to the paver surface, sweeping it across and filling the paver joints.
6. Remove excess aggregate on the surface by sweeping the pavers clean.
7. Compact and seat the pavers into the bedding material using a low-amplitude, 75–90 Hz plate compactor capable of at least 22 kN. Make at least two passes in perpendicular directions with the plate compactor.
8. Apply additional aggregate to the openings and joints if needed, filling them completely.
9. Remove excess aggregate by sweeping.
10. All pavers within 2 m of the laying face must be left fully compacted at the completion of each day.
11. The final surface tolerance of compacted pavers shall not deviate more than ±10 mm under a 3 m straightedge.
12. The surface elevation of pavers shall be 3 to 6 mm above adjacent drainage inlets, concrete collars or channels.

3.5 FIELD QUALITY CONTROL

A. After sweeping the surface clean, check final elevations for conformance to the drawings.

B. Lippage: No greater than 3 mm difference in height between adjacent pavers.

Note: The surface of the pavers may be 3 to 6 mm above the final elevations after compaction. This height helps compensate for possible minor settling normal to pavements.

C. The surface elevation of pavers shall be 3 to 6 mm above adjacent drainage inlets, concrete collars, or channels.

D. Bond lines for paver courses: ±15 mm over a 15 m taut string line.

E. Verify the surface infiltration at a minimum of 250 cm/h using test method ASTM C1781.

3.6 PROTECTION

A. After work in this section is complete, the general contractor shall be responsible for protecting work from sediment deposition and damage caused by subsequent construction activity on the site.

B. The PICP installation contractor shall return to the site after six months from the completion of the work and provide the following as required: filling paver joints with stones, replacing broken or cracked pavers, and releveling settled pavers to initial elevations. Any additional work shall be considered part of the original bid price, with no additional compensation.

APPENDIX C

EXAMPLES OF ORIFICE AND COMMON WEIR EQUATIONS

Several methods exist to calculate the outflow rate (Q_U) from permeable pavement control structure outlets. Several common methods to calculate flow for free-flowing hydraulic structures are detailed below. The engineer should consult a relevant hydraulic reference manual or textbook to understand the necessary assumptions and flow conditions that make each equation applicable.

Weirs

Weirs are elevated structures in open channels or control structures that regulate flow. In these structures, water flows through an opening of regular shape (typically rectangular, triangular, or trapezoidal) in which the relationship between flow rate and depth is fixed and known. The bottom edge of the opening is called the crest, and there are two common types of weirs:

1. Sharp-crested weirs, and
2. Broad-crested weirs.

1. Sharp-Crested Weirs

Sharp-crested weirs, or thin-plate weirs, are a thin metallic or plastic plate set vertically and perpendicular to the flow. The main type of sharp-crested weirs are rectangular and V-notch weirs. Limitations include the following:

a. To be considered sharp-crested, the thickness of the plate should be between 1 and 2 mm (40 and 80 mil) (Martinez et al. 2005).

b. If plates are thicker than specified in item a, the plated edge must be beveled a minimum of 45°. For V-notch weirs, a 60° bevel is recommended.

c. To have “end contractions,” the weir must span only a portion of the outflow channel or outlet structure to permit atmospheric conditions beneath the weir nappe (the sheet of water flowing over the weir). Weirs that span the entire outflow channel or outlet are considered to be suppressed (or uncontracted).

Sharp-Crested Rectangular Weirs with End Contractions

$$Q_U = C \times (L - 0.1nH) \times H^{3/2} \qquad \text{(C-1)}$$

where

Q_U = Flow rate, in m^3/s (ft^3/s);

H = Head on the weir, in m (ft);

L = Crest length of the weir, in m (ft);

n = Number of sides on the weir that are contracted (typically 2); and

C = Weir coefficient (typical value = 1.83) (Chin 2006).

Sharp-Crested Rectangular Weir without End Contractions

$$Q_U = C \times L \times H^{3/2} \qquad \text{(C-2)}$$

where

Q_U = Flow rate, in m^3/s (ft^3/s);

H = Head on the weir, in m (ft);

L = Crest length of the weir, in m (ft); and

C = Weir coefficient (typical value = 1.83) (Chin 2006).

V-Notch Weirs

$$Q_U = \frac{8}{15} C \times \sqrt{2g} \times \tan\frac{\theta}{2} \times H^{5/2} \qquad \text{(C-3)}$$

where

Q_U = Flow rate, in m^3/s (ft^3/s);

H = Head above the weir invert or notch, in m (ft);

g = Acceleration caused by gravity, 9.81 m/s^2 (32.2 ft/s^2);

θ = Notch angle (typically between 10° and 90°); and

C = Weir coefficient, which varies by notch angle (θ) and head (H):

= 2.50 for 90°,

= 1.44 for 60°, and

= 1.03 for 45°.

2. Broad-Crested Weirs

Broad-crested weirs (or long-based weirs) have significantly longer crest lengths as compared to sharp-crested weirs and are typically constructed of concrete or wood, have rounded edges, and are capable of managing much larger discharge rates. Designs of broad-crested weirs vary significantly. A rectangular broad-crested weir is the most common and its flow rate is presented here:

$$Q_U = C \times \sqrt{gL} \times \left(\frac{2}{3}H\right)^{3/2} \qquad \text{(C-4)}$$

where

Q_U = Flow rate, in m^3/s (ft^3/s);

H = Head, which is the energy of the upstream flow measured relative to the weir crest elevation, in m (ft):

$H = h_1 + \frac{{v_1}^2}{2g}$; where h_1 = Elevation of the upstream water surface above the weir crest and v_1 = Average velocity of flow upstream of the weir;

g = Acceleration caused by gravity, 9.81 m/s^2 (32.2 ft/s^2);

L = Length of the weir, in m (ft); and

C = Weir coefficient:

$C = \frac{0.65}{(1+H/H_W)^{1/2}}$ where H_w = Height above weir crest and height above the channel invert.

Orifice or Tube Orifice

Flow through an orifice is synonymous with a culvert that has inlet control or USGS Type 5 flow condition; that is, the inlet is submerged and the outlet is unsubmerged. The head–discharge relationship in this case is based on orifice flow for a submerged inlet. Once the inlet is submerged, the head–discharge relationship is based on orifice flow for a submerged inlet, and the governing hydraulic equation is the orifice flow equation. Use of an orifice is the most commonly applied flow regulation technique for permeable pavement applications.

$$Q_U = C \times A \times \sqrt{2g\Delta h} \qquad \text{(C-5)}$$

where

Q_U = Flow rate, in m^3/s (ft^3/s);

A = Cross-sectional area of the inlet or orifice, in m^2 (ft^2);

Δh = Differential head measured from the centroid of the orifice, in m (ft);

g = Acceleration caused by gravity, 9.81 m/s^2 (32.2 ft/s^2);

C = Coefficient of discharge; and

Orifice = C = 0.63 for squared-edge entrances (Chin 2006);
C = 1.0 for well-rounded entrances (Franzinni and Finnemore 2002); and
C = 0.80 for an orifice tube (Lake Simcoe Region Conservation Authority 2017).

Flow in Pipes

For permeable pavement designs where a submerged inlet condition is not anticipated (see orifice or orifice tube), the outflow from the system can be calculated as flow in pipes using two common methods:

1. Hazen–Williams formula, and
2. Manning's equation.

1. Hazen–Williams Formula

The equation is applicable to the flow of water at 16°C in pipe diameters ranging from 50 mm to 1,850 mm (2 to 72 in.) and flow velocities less than 3 m/s (10 ft/s) (Mott 1994).

$$V = 0.849 C_H R^{0.63} {S_f}^{0.54} \quad \text{(C-6.si)}$$

$$V = 1.318 C_H R^{0.63} {S_f}^{0.54} \quad \text{(C-6)}$$

where

V = Flow velocity, in m/s (ft/s);

C_H = Hazen–Williams roughness coefficient dependent on pipe materials;

R = Hydraulic radius, in m (ft);

S_f = Slope of energy grade line:

$S_f = \frac{h_f}{L}$; where h_f = Head loss caused by friction over the pipe length L. Note: It is common to substitute pipe slope for S_f; and

2. Manning's Equation

Manning's equation applies only to rough turbulent flow, where the frictional head losses are controlled by the relative roughness.

$$V = \frac{1.486}{n} R^{2/3} {S_f}^{1/2} \quad \text{(C-7.si)}$$

$$V = \frac{1}{n} R^{2/3} {S_f}^{1/2} \quad \text{(C-7)}$$

where

V = Flow velocity, in m/s (ft/s);

n = Manning roughness coefficient, dependent on pipe materials;

R = Hydraulic radius, in m (ft);

S_f = Slope of energy grade line:

$S_f = \frac{h_f}{L}$; where h_f = Head loss caused by friction over the pipe length L. Note: It is common to substitute pipe slope for S_f; and

APPENDIX D

PICP STRUCTURAL DESIGN USING AASHTO 1993, *GUIDE FOR DESIGN OF PAVEMENT STRUCTURES*

As an alternative to the design procedure in this standard, the following procedure and assumptions may be used as an alternative structural design method. The structural design procedure incorporates the reliability design concepts from AASHTO (1993). This guide does not include structural design equations for permeable pavements. However, it was widely used before the development of the University of California Pavement Research Center (UCPRC 2014) structural design approach. Modifications were made to the design assumptions using the flexible pavement structural design equation in the *Guide* to produce Table D-1 (Smith 2017) for use on PICP structural designs.

Design Assumptions

One of several assumptions in the flexible design procedure says that the higher the selected reliability and standard deviation, the higher the ESALs used in the design. The effect of the reliability and standard deviation are factored from the actual ESALs using the following equation:

$$\text{Log (Design ESALs)} = \log\text{ (Actual ESALs)} + Z_R \times S_o \qquad \text{(D-1)}$$

For Table D-1, a constant reliability level of 80% ($Z_R = 0.841$) and standard deviation, $S_o = 0.44$ were selected to produce conservative subbase thicknesses. Using the above equation and an actual ESAL value of 400,000, the reliability function of the AASHTO design equation would result in a factored ESAL value of 800,000 ESALs.

In addition, the following design parameters according to the AASHTO equation were used in the development of the structural design table for pedestrian and vehicular traffic for various subgrade and traffic conditions shown in Table D-1:

W = Variable;
Z_R = 0.841 for R = 80%;
S_o = 0.44;
a_i = Structural layer coefficients used in this standard are as follows:
80 mm (3 1/8-in.) concrete paver and 50 mm (2-in.) ASTM No. 8 bedding layer = 0.3,
ASTM No. 57 base layer = 0.09, and
ASTM No. 2 subbase layer = 0.06;
m_i = Drainage coefficient = 1.0;

p_i = 4.2;

p_t = 2.5; and

M_R = Soil subgrade stiffness is variable depending on site conditions.

M_R in psi = 2,555 × $\text{CBR}^{0.64}$; M_R in MPa = 17.61 × $\text{CBR}^{0.64}$, and

M_R in psi = 1,155 + 555 × R; M_R in MPa = (1,155 + 555 × R)/145.

Table D-1A. PICP Structural Design Chart for Pedestrian and Vehicular Applications (Metric Units)

Use/Parameter	Soaked CBR (%)	3	4	5	6	7	8	9	10
	R-Value	7.5	9	11	12.5	14	15.5	17	18
	Resilient Modulus (MPa)	36	43	49	55	61	67	72	77
			All thickness values in millimeters						
Pedestrian Only	Base	150	150	150	150	150	150	150	150
	Subbase	0	0	0	0	0	0	0	0
Occasional Service/ Emergency Vehicles	Base	100	100	100	100	100	100	100	100
	Subbase	150	150	150	150	150	150	150	150
ESALs (TI)									
50,000 (6.3)	Base	100	100	100	100	100	100	100	100
	Subbase	175	150	150	150	150	150	150	150
100,000 (6.8)	Base	100	100	100	100	100	100	100	100
	Subbase	275	200	150	150	150	150	150	150
200,000 (7.4)	Base	100	100	100	100	100	100	100	100
	Subbase	425	325	275	225	175	150	150	150
300,000 (7.8)	Base	100	100	100	100	100	100	100	100
	Subbase	500	400	350	300	250	225	200	175
400,000 (8.1)	Base	100	100	100	100	100	100	100	100
	Subbase	550	475	400	350	300	275	250	225
500,000 (8.3)	Base	100	100	100	100	100	100	100	100
	Subbase	600	525	450	400	350	300	275	250
600,000 (8.5)	Base	100	100	100	100	100	100	100	100
	Subbase	650	550	475	425	375	350	300	275
700,000 (8.6)	Base	100	100	100	100	100	100	100	100
	Subbase	700	600	525	450	425	375	350	300
800,000 (8.8)	Base	100	100	100	100	100	100	100	100
	Subbase	725	625	550	500	450	400	375	325
900,000 (8.9)	Base	100	100	100	100	100	100	100	100
	Subbase	750	650	575	525	475	425	400	350
1,000,000 (9)	Base	100	100	100	100	100	100	100	100
	Subbase	775	675	600	525	475	425	400	375

Notes: CBR = California bearing ratio. Base and subbase thickness values are based on the assumptions outlined following.

Base = ASTM No. 57 aggregate; Subbase = ASTM No. 2 aggregate. Similar-sized aggregates may be used for the base and subbase.

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.

Table D-1B. PICP Structural Design Chart for Pedestrian and Vehicular Applications (U.S. Customary Units)

Use/Parameter	Soaked CBR (%)	3	4	5	6	7	8	9	10
	R-Value	7.5	9	11	12.5	14	15.5	17	18
	Resilient Modulus (psi)	5,220	6,205	7,157	8,043	8,877	9,669	10,426	11,153
		All thickness values in inches							
Pedestrian Only	Base	150	150	150	150	150	150	150	150
	Subbase	0	0	0	0	0	0	0	0
Occasional Service/ Emergency Vehicles	Base	4	4	4	4	4	4	4	4
	Subbase	6	6	6	6	6	6	6	6
ESALs (TI)									
50,000	Base	4	4	4	4	4	4	4	4
(6.3)	Subbase	7	6	6	6	6	6	6	6
100,000	Base	4	4	4	4	4	4	4	4
(6.8)	Subbase	11	10	6	6	6	6	6	6
200,000	Base	4	4	4	4	4	4	4	4
(7.4)	Subbase	17	13	11	9	7	6	6	6
300,000	Base	4	4	4	4	4	4	4	4
(7.8)	Subbase	20	16	14	12	10	9	8	7
400,000	Base	4	4	4	4	4	4	4	4
(8.1)	Subbase	22	19	16	14	12	11	10	9
500,000	Base	4	4	4	4	4	4	4	4
(8.3)	Subbase	24	21	18	16	14	12	11	10
600,000	Base	4	4	4	4	4	4	4	4
(8.5)	Subbase	26	22	19	17	15	14	12	11
700,000	Base	4	4	4	4	4	4	4	4
(8.6)	Subbase	28	24	21	18	17	15	14	12
800,000	Base	4	4	4	4	4	4	4	4
(8.8)	Subbase	29	25	22	20	18	16	15	13
900,000	Base	4	4	4	4	4	4	4	4
(8.9)	Subbase	30	26	23	21	19	17	16	14
1,000,000	Base	4	4	4	4	4	4	4	4
(9)	Subbase	31	27	24	21	19	18	16	15

Notes: CBR = California bearing ratio. Base and subbase thickness values are based on the assumptions outlined below.

Base = ASTM No. 57 aggregate; Subbase = ASTM No. 2 aggregate. Similar-sized aggregates may be used for the base and subbase.

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.

3 1/8 in.-thick concrete pavers and 2 in. ASTM No. 8 bedding layer coefficient = 0.3.

APPENDIX E

APPROXIMATE CORRELATION BETWEEN PERMEABILITY AND UNIFIED SOIL CLASSIFICATION

USCS Soil Classification per ASTM D2487 (2011a)	*Coefficient of Permeability,* k, *approximate μm/s (in./h)*
GW–Well-graded gravels	9.12 to 896 (1.3 to 127)
GP–Poorly graded gravels	50 to 96,661 (6.8 to 13,700)
GM—Silty gravels	0.00092 to 95 (1.3×10^{-4} to 13.5)
GC—Clayey gravel	9.17×10^{-5} to 0.0917 (1.3×10^{-5} to 1.3×10^{-2})
SW—Well-graded sands	5 to 480 (0.7 to 68)
SP—Poorly graded sands	0.49 to 4.9 (0.07 to 0.7)
SM—Silty sands	0.00091 to 4.9 (1.3×10^{-4} to 0.7)
SC—Clayey sands	9.17×10^{-5} to 4.9 (1.3×10^{-5} to 0.7)
ML—Inorganic silts of low plasticity	9.17×10^{-5} to 4.9 (1.3×10^{-5} to 0.7)
CL—Inorganic clays of low plasticity	9.17×10^{-5} to 0.00917 (1.3×10^{-5} to 1.3×10^{-3})
OL—Organic silts of low plasticity	9.17×10^{-5} to 0.0917 (1.3×10^{-5} to 1.3×10^{-2})
MH—Inorganic silts of high plasticity	9.17×10^{-6} to 9.17×10^{-4} (1.3×10^{-6} to 1.3×10^{-4})
CH—Inorganic clays of high plasticity	9.17×10^{-7} to 9.17×10^{-5} (1.3×10^{-7} to 1.3×10^{-5})
OH—Organic clays of high plasticity	Not appropriate under permeable pavements
PT—Peat, mulch, and soils with high organic content	Not appropriate under permeable pavements

Note: These values characterize the permeability of rolled-earth embankments with moisture-density control. They are from Moulton (1980). The table originally was published in Sherard et al. (1963). These values can be compared to those in Table 4-9 to better understand the effect of compaction on soil permeability.

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