# A NEW URBAN TREE SOIL TO SAFELY INCREASE ROOTING VOLUMES UNDER SIDEWALKS

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**Abstract.** Soil compaction, which is necessary to safely support sidewalks and pavement, conflicts with urban trees' need for usable rooting space to support healthy tree growth. We have defined a rigid soil medium that will safely bear loads required by engineering standards yet still allow for rapid root exploration and growth. This was accomplished by forming a stone matrix and suspending soil within the matrix pores with the assistance of a hydrogel gluing agent. Initial studies using three stone types and various stone to soil ratios showed that the compacted stone–soil test medium (dry densities > 1700 kg/m$^3$) increased root growth by a minimum of 320% over the compacted clay loam control (dry density of 1378 kg/m$^3$). The proposed system can safely bear load demonstrated by California Bearing Ratios consistently exceeding 40. Discussion of a critical mixing ratio is presented as an approach for developing a specification for field installation.

Because lack of rooting space is arguably the most limiting factor affecting a street tree's water and nutrient demands over time, urban trees need to have access to larger volumes of soil if they are to achieve the size, function, and benefits for which we plant them [13, 17]. Urban soil compaction generally occurs in what would be the tree's preferential rooting zone: the shallow lens of soil no more than three feet deep extending well beyond the tree's canopy [18]. Compaction contributes to insufficient rooting volumes by increasing the soil's bulk density and soil strength to levels which impede root growth [3,8,10,25].

While several reasons for densification and compaction of urban soils exist, the most ubiquitous problem we face is the purposeful compaction of the soil surrounding a street tree to support pavement or nearby structures. Compaction is necessary as a cost–effective way to increase the strength and stability of existing soil materials to prevent their settlement under or around designed structures [7,11,14,23]. It increases the bearing capacity of the materials below the pavement system and reduces the shrinking and swelling of soils that occur with water movement or frost action [11,26]. Thus any effort to increase the rooting area for street trees under pavement must accept the necessity of compaction and understand the levels of compaction needed to safely design pavement structures.

**Proctor density.** A standard measure of compactive effort which is often specified is termed Proctor density. It is important to understand exactly what this term means and how it is identified. Originally developed for evaluating and controlling compaction of fine textured soils when building earthen dams, the Proctor testing procedure describes the relationship between soil moisture, a standard compactive effort, and soil porosity (void space)[19,20,21,22]. As the soil moisture increases, a standard compactive effort yields progressively greater bulk densities (and fewer voids) up to an optimum. After the optimum is reached, densities decrease with increasing soil moisture because the soil is held apart by the incompressible excess water in the test sample [19,21]. Proctor Optimum Density is the high point on a curve plotting the dry density of the soil against increasing moisture content as a result of a standard compactive effort arbitrarily set to simulate a compaction effort used in the field (Figure 1)[2,12]. The standard effort consists of the near equivalent of 25 blows from a 5.5 lb hammer falling 1 foot onto each of 3 equal layers of material. These layers fill a 4 inch diameter, 4.584 inch depth mold in the usual test [2]. By adjusting the moisture content to match that associated with the observed peak of the moisture density curve, a contractor can get the most efficient compaction per unit effort. Compaction at other levels of moisture content would require more labor to reach the density achieved by

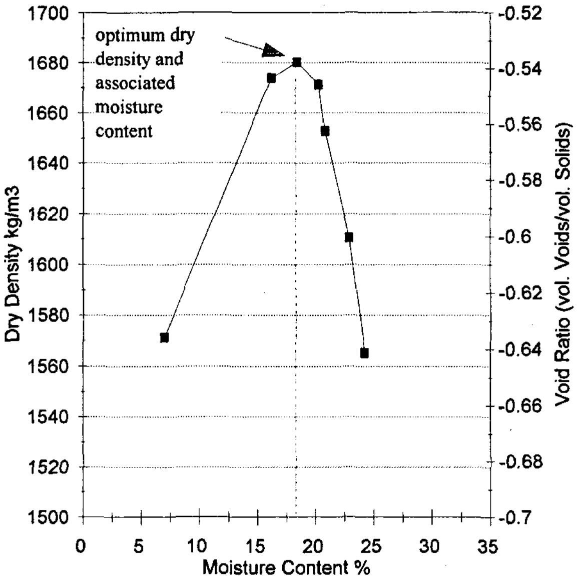



**Figure 1. Curve showing the moisture density relationship found for a clay loam soil as the result of a standard Proctor compaction effort. The peak of this curve would be defined as 100% standard Proctor density. The effects of increased density on the porosity of the soil is also shown via the void ratio. Porosity = void ratio / (1 + void ratio).**

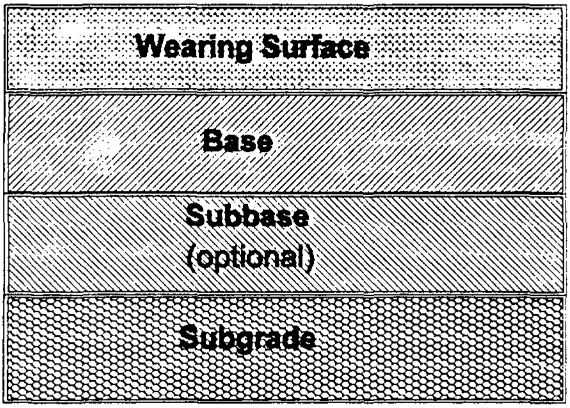
compacting at the optimum moisture content and could affect the compacted strength of the material despite an acceptable dry density [11]. A lesser density will usually have an associated lower strength and bearing capacity [11].

The density generated by the previously described laboratory test at optimum moisture content is defined as 100% standard Proctor Density and serves as a benchmark set to maintain quality control over the compaction process during construction. In practice, construction specifications require a percentage of Proctor Density and the term "Proctor Density" could be derived from any of the compactive efforts described by the ASTM moisture density relationship specifications [2]. Since the test is standardized, it is often used to generate a dry density from which to test a material's shear strength, bearing capacity, and/or deflection resistance. This information can be used to evaluate and define a material for safe engineering design practices. In a sidewalk or parking situation, a failure could translate into large financial liabilities such as vehicle damage, personal injury, increased maintenance, or premature replacement costs.

**California Bearing Ratio.** The Proctor moisture/density relationship is also used to identify a standardized testing point for evaluating a material's load bearing capacity via the California Bearing Ratio (CBR) [1,26]. This ratio compares materials used under pavements to a standard material which has been empirically determined to be a satisfactory pavement base [2,11]. This value is dependent on frictional strength, therefore moisture content and bulk density are major factors in this testing procedure. A CBR value of 100 would be interpreted to mean that the tested material had the same bearing capacity as the reference standard (100%).

With the CBR value, the necessary pavement thickness can be determined by evaluating components of the soil profile materials for shear strength under pavement [1,5,27]. A typical soil profile under pavement would include the subgrade, i.e. a native or otherwise preexisting soil typically with a large amount of fine particles. The subbase and/or base courses are usually well–graded gravels, and the wearing surface is what we often think of as pavement (Figure 2) [11]. Acceptable CBR values are assigned for each layer used in pavement systems with minimum acceptable bearing capacities increasing for each consecutive layer from the bottom toward the top surface grade. The subgrade, which is the deepest level, often has a comparatively low CBR in the range of 5 to 10 [1,11]. Base materials are normally much stronger than the subgrade with acceptable CBR values ranging from 40 to 80 [11]. These values could be considered acceptable for materials used under paved surfaces in light traffic situations which would include maintained municipal sidewalks.

The CBR test places a compacted cylinder of material onto a loading press that forces a piston into the soil to a depth of 0.5 inches at a uniform rate. The strength of the material is found by measuring the load required to continue the penetration. A curve plotting load against penetration is shown in Figure 3. This curve is corrected for surface irregularities as shown by the seg-




**Wearing Surface**: *Often concrete or asphalt*, depth is dependent on the material used.

**Base**: A very stable layer which is a sand-gravel material or a material stabilized with a binding agent. This layer can be from 5 - 30 cm in depth.

**Subbase**: An optional layer of stable material, often 15 - 30 cm of a sandy or gravelly material.

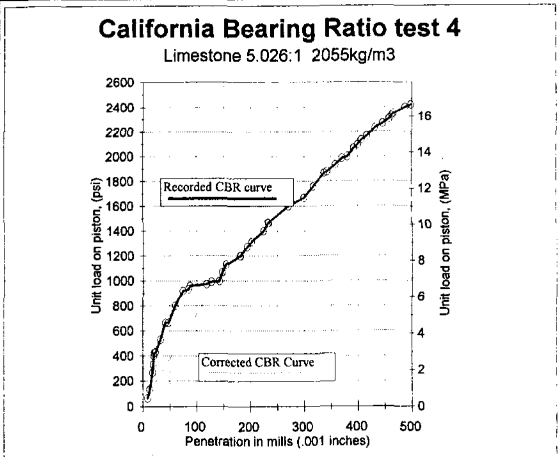
**Subgrade**: The preexistent soil at the site. The top layer is compacted before any of the base or pavement layers are installed.

**Figure 2. Definition and locations of the layers which can make up a sidewalk. Adapted from Holtz and Kovaks (11).**

mented line. The point where the segmented line intersects the X axis is defined as the corrected starting point and the segmented line is used to describe the load/penetration relationship [2]. The load at 100 mills (0.01 inch) on the corrected curve is divided by the load needed for the same penetration into the standard reference material (6.89 MPa) [2, 26]. The resultant value expressed as a percentage is the CBR value.

A marginally acceptable or unacceptable base could have an acceptable CBR at field capacity and lower moisture levels normally found outside of the laboratory, but could fail in a saturated condition, which often occurs in the spring. For this reason, CBR tests are often subjected to a 96 hour saturation period to accommodate the worst case scenario.

**Soil classification systems.** Horticulturists and soil scientists often use the USDA soil classification system for characterizing agricultural soil




| Penetration | | Recorded load | | Corrected load | | Standard load | | Observed | Corrected |
|---|---|---|---|---|---|---|---|---|---|
| mills | mm | psi | MPa | psi | MPa | psi | MPa | CBR | CBR |
| 25 | 0.6 | 444 | 3.06 | 267 | 1.84 | | | | |
| 50 | 1.3 | 679 | 4.68 | 467 | 3.22 | | | | |
| 75 | 1.9 | 921 | 6.35 | 610 | 4.21 | | | | |
| 100 | 2.5 | 965 | 6.66 | 783 | 5.40 | 1000 | 6.89 | 96.60 | 78.37 |
| 125 | 3.2 | 994 | 6.85 | 1297 | 8.94 | | | | |
| 150 | 3.8 | 1094 | 7.54 | 1050 | 7.24 | | | | |
| 175 | 4.4 | 1187 | 8.18 | 1167 | 8.04 | | | | |
| 200 | 5 | 1304 | 8.99 | 1290 | 8.89 | 1500 | 10.34 | 86.97 | 86.02 |
| 300 | 7.5 | 1667 | 11.49 | 1717 | 11.84 | 1900 | 13.1 | 87.72 | 90.35 |
| 400 | 10 | 2093 | 14.43 | 2093 | 14.43 | 2300 | 15.86 | 91.00 | 91.00 |
| 500 | 12.5 | 2420 | 16.69 | 2420 | 16.69 | 2600 | 17.93 | 93.06 | 93.06 |

**Figure 3. A typical CBR test from the tested limestone stone/soil mix reported in graphic and numeric form. The CBR value is calculated by dividing the tested load by the standard load and multiplying the result by 100.**

systems and describe their behavior in terms of their porosity, nutrient holding capacity, and drainage [30]. The geotechnical engineering community uses the Unified Classification System to characterize materials. This system could be called a classification by behavior during engineering uses with divisions in classification coinciding with shifts in engineering characteristics [7,14,28]. We felt that the soil mixes we were developing should be defined using the Unified Classification System to better communicate our most promising soil mixes to the engineering community.

Geotechnical engineers use the Unified Classification System to help predict soil properties such as frost-heave susceptibility, drainage and water infiltration, expected compacted densities, bearing strength, and pavement base efficacy [5,28,29]. Figure 4 shows how a material's clas-

| MAJOR DIVISIONS | | SYMBOL | | NAME | VALUE AS SUBBASE WHEN NOT SUBJECT TO FROST ACTION | VALUE AS BASE WHEN NOT SUBJECT TO FROST ACTION | POTENTIAL FROST ACTION | DRAINAGE CHARACTERISTICS | TYPICAL CBR VALUE |
|---|---|---|---|---|---|---|---|---|---|
| COARSE-GRAINED SOILS | GRAVEL AND GRAVELLY SOILS | GW | | Well-graded gravel or gravel-sand mixtures, little or no fines | Excellent | good | none to very slight | Excellent | 40 - 80 |
| | | GP | | Poorly graded gravels or gravel-sand mixtures little or no fines | good | fair to good | none to very slight | Excellent | 30 - 60 |
| | | GM | d | silty gravels gravel-sand-silt mixtures | good | fair to good | slight to medium | fair to poor | 40 - 60 |
| | | | u | | fair | poor to not suitable | slight to medium | poor to practically impervious | 20 - 30 |
| | | GC | | Clayey gravels gravel sand-clay-mixtures | fair | poor to not suitable | slight to medium | poor to practically impervious | 20 - 30 |
| | SAND AND SANDY SOILS | SW | | Well-graded sands or gravelly sands little or no fines | fair to good | poor | none to very slight | Excellent | 20 - 40 |
| | | SP | | Poorly graded sands or gravelly sands, little or no fines | fair | poor to not suitable | none to very slight | Excellent | 10 - 40 |
| | | SM | d | Silty sands, sand-silt mixtures | fair to good | poor | slight to high | fair to poor | 15 - 40 |
| | | | u | | poor to fair | not suitable | slight to high | poor to practically impervious | 10 - 20 |
| | | SC | | Clayey sands, sand-clay mixtures | poor | not suitable | slight to high | poor to practically impervious | 5 - 20 |

**Figure 4. Material behavior as related to the Unified Classification system. Adapted from Holtz and Kovaks (11) and US Waterway Station (28).**

sification can roughly predict performance as a pavement base. The last column notes typical CBR ranges for listed materials. Note that for a marginally acceptable base (CBR = 40), a formidable load of 2.76 MPa (400 psi) is required to penetrate 2.5 mm (0.1 inches). Since the CBR can be affected by moisture in fine grained soils, a CBR of 40 would normally be marginally acceptable in a saturated condition, and penetration resistance would increase as the soil dried. Luckily, root tips are much smaller than a CBR testing piston and may find small zones of less resistance, but the example serves to highlight the contradictory demands of root expansion and base compaction for sidewalks.

**Implications for roots.** By acknowledging the need for compaction from a structural viewpoint, we can understand why roots often have trouble penetrating the bases and subgrades of many sidewalks. For sidewalks, a minimal removal of existing material is often the case, so the subgrade very often will lie within 8 inches of the final grade. This is the first zone that experiences a compactive effort during construction. Base materials are placed onto the subgrade and compacted as well [11]. Both the subgrade and the base are normally compacted to at least 95% of an optimum density, which often is restrictive to root growth [11,15,18].

It is thus no surprise that when roots "escape" or outgrow their planting holes they usually choose zones of lesser compaction due to sub–surface structures such as along utility lines, or the base course immediately beneath the actual pavement where the open granular nature of the layer might contain enough voids to allow root growth [6,15,17,18]. It is also of little surprise to observe sidewalk damage from those roots which expand radially as they grow directly beneath the pavement since this interface can provide greater opportunity for root penetration and growth in comparison with the compacted layers below.

Street trees prefer a less dense rooting medium that allows roots to penetrate to a depth of two to three feet, but this is currently unacceptable under sidewalks from a structural safety viewpoint. Those trees that do not "break out" are sentenced to a limited future dictated by the limited amount of designed rooting volume within the planting pit or island. This volume is not likely to support the tree for the designer's and the public's expected life span as borne out by the high tree mortality rate found in planting areas surrounded by pavement; often dying in as little as 7 years [13,16].

## A New System

To solve this problem, our objective was to develop an easily produced soil medium that would meet engineers' specifications for load bearing capacity, but still allow for vigorous root growth through the compacted profile, thus increasing overall rooting volume without compromising safety. This could be thought of as an evolution of the compaction-resistant planting medium employed by Patterson in Washington, D.C. but with greater load bearing requirements [16]. Our system would build a gap–graded, load bearing stone matrix that could meet the engineering requirements while suspending a noncompacted rooting medium within the voids that exist between the stones.

Materials in which the full range of particle size classes are lacking except for one or two widely varying size classes are often termed "gap-graded." Figure 5 shows examples of the gap-graded materials we have tested. These materials exhibit good drainage capabilities due to the inability of the particles to tightly nest into a uniform soil profile. It was our intent to use this fact to our advantage in manufacturing such a medium.

In our medium, gravel and soils were mixed so that loads would be transferred from stone to stone in the gravel while leaving the soil between the stones essentially unaffected by compaction. Theoretically, roots would be encouraged to grow deeper into the uncompacted soil between the stones which allow for greater water and air movement. This medium might also reduce sidewalk failure, another goal of this system, by encouraging deeper root systems.

A series of studies were initiated to identify a

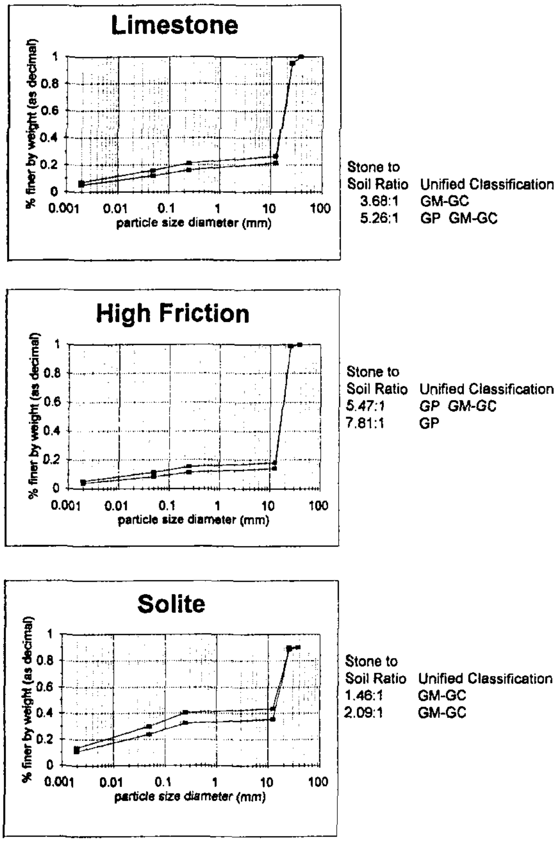



**Figure 5. Linden test media particle size distributions. All curves represent the extremes of the tested stone to soil ratios. In each graph, the higher curve represents the highest tested ratio, the lower curve represents the lowest tested ratio. Note the gap graded nature of the mixes with material between 25 mm and 12 mm all but lacking entirely.**

promising stone to soil ratio that would meet our objectives. To achieve this end, two important principles had to be recognized. First, to prevent soil compaction and facilitate the necessary air-filled porosity, the volume of soil in the stone and soil mix must be less than the total porosity of the compacted stone matrix. At this point, the bearing capacity of the system would largely become a function of the strength of the stone alone. The determination of this point was a critical step in the definition of this medium. Second, the soil could not be allowed to sieve to the bottom of the stone matrix during the mixing or compaction phases of its installation. A small amount of a hydrated hydrogel was added to the stone matrix before blending in the soil to prevent the stone and soil from separating. This hydrogel acted as a glue, attaching the soil to the stone much as a tackifier works in hydroseeding applications.

## Materials and Methods

**Linden study.** The three types of stone chosen for the initial tests are described in Table 1. Crushed limestone was chosen for its angularity and consistency as a manufactured material. A high frictional quarried gravel was chosen for its predominantly round shape. A third stone type, Solite® (a heat expanded slate), was chosen for its rigid nature, light weight, and porosity. The crushed and quarried stones conformed to a 0.5 – 1.0 inch gravel size range which was purchased as a #2 size stone [2]. A clay loam was chosen for the interstitial soil component of the mix because of its water and nutrient holding capacity, a critical factor in a mostly stone root environment. Twelve blends were used in this first test representing four increasing stone: soil volumetric ratios for each of the three stone types.

To determine stone to soil ratios, the percentage of voids within a matrix of each stone type were measured. Five random samples from each stone type were placed into containers of known volume and brought to a saturated, surface dry, condition. From this point, a loose pack porosity was determined for each stone type by measuring the amount of water needed to fill the container containing the stone sample. The mean of five measurements determined the noncompacted matrix porosity of 44.7% for the crushed limestone, 40.0% for the gravel, and 47.8% for the Solite®.

For each stone type, four mixes were generated by adding enough clay loam to fill 100, 90, 80, and 70% of the measured noncompacted porosity. The resultant blends are listed using a dry weight ratio as shown in Table 2. The dry weight ratios varied due to the different specific gravities of each stone type. The Unified Soil Classification System was used to define each blend and to predict their performance in an engineering context.

Each of these blends was also blended with a poly–acrylimide hydrogel tackifier (Gelscape® Amereq Corporation) to prevent aggregate separation during the mixing and compaction of such gap–graded mixes (Figure 4). The tackifier was used at a rate of 38 grams per 13650 $cm^3$ of uncompacted mix (approximately 152 grams hydrogel per 100 kg stone on a dry weight basis). Compacted clay loam with and without hydrogel served as the controls for a total of 26 treatments with six replications.

Each mixture was blended in a small rotary concrete mixer in two batches and then combined. For each blend, six 14.2 L nursery containers ( #5 short) were filled for a single lift compaction. Excess material was stored to fill settlements after the initial compactive effort. The containers were

**Table 1. Materials used in developing test blends. The Solite® was blended to approximate the same particle size distribution as the other two stone types.**

| Material used | Specific gravity (Gs) | % passing 38.1 mm sieve (1.5") | % passing 25.4 mm sieve (1.0") | % passing 12.7 mm sieve (0.5") | Coefficient of uniformity (Cu) | Description |
|---|---|---|---|---|---|---|
| #2 Crushed limestone | 2.71 | 100 | 94.1 | 6.7 | 1.4 | All angular stone |
| #2 High friction aggregate | 2.66 | 100 | 98.8 | 3.2 | 1.34 | Round quarried gravel,oversized crushed and blended back, % limestone = 16% |
| Solite® | 1.50 | > 90 | 83 | < 5 | 2 | Exploded slate, very porous |
| Soil | 2.58 | — | 26.4 % sand<br>40% silt<br>33.6% clay<br>(USDA) | — | — | Shredded clay loam, pH = 5.25<br>- dry bulk density = 1110 kg/m$^3$<br>- Plastic limit = 20.5, liquid limit = 27.5<br>- Std. Proctor opt. density = 1674 kg/m$^3$ |

**Table 2. Description of linden study media. Densities were measured at the end of the study. Overall standard error of density by treatment = 24.69 kg/m³ excepting where single replicates had died (X); in which case the standard error = 27.04 kg/m³.**

| Stone type | Stone to soil dry weight ratio | Calculated Proctor optimum density (kg/m³) | Calculated porosity (%) at Proctor optimum density | Observed dry density in kg/m³ (% Proctor optimum) | | % Actual porosity | |
|---|---|---|---|---|---|---|---|
| | | | | without hydrogel | with hydrogel | without hydrogel | with hydrogel |
| Limestone | 368:1 | 2000 | 25.5 | 1789 (89%) | 1594 (80%) | 33.4 | 40.6 |
| | 4.09:1 | 1987 | 26.0 | 1767 (89%) | 1571 X (79%) | 34.2 | 41.5 |
| | 4.60:1 | 1978 | 26.4 | 1748 (88%) | 1602 (81%) | 35.0 | 40.4 |
| | 5.26:1 | 1965 | 27.0 | 1638 X (83%) | 1594 X (81%) | 39.1 | 40.8 |
| High friction | 5.47:1 | 2068 | 21.9 | 1823 (88%) | 1681 (81%) | 31.2 | 36.5 |
| | 6.03:1 | 2038 | 23.1 | 1812 (89%) | 1692 (83%) | 31.6 | 36.2 |
| | 6.84:1 | 2004 | 24.4 | 1852 (92%) | 1716 (86%) | 30.1 | 35.3 |
| | 7.81:1 | 1972 | 25.6 | 1784 (90%) | 1723 (87%) | 32.7 | 35.0 |
| Solite® | 1.46:1 | 1500 | 22.8 | 1269 (85%) | 924 (62%) | 34.7 | 52.4 |
| | 1.70:1 | 1414 | 25.7 | 1216 (86%) | 1132 (80%) | 36.1 | 40.5 |
| | 1.78:1 | 1391 | 26.5 | 1226 (88%) | 1154 (83%) | 35.2 | 39.0 |
| | 2.09:1 | 1317 | 28.9 | 1153 X (88%) | 1122 (85%) | 37.8 | 39.4 |
| Clay loam (Ck) | — | 1674 | 35.4 | 1378 X (82%) | 1248 (75%) | 46.8 | 51.8 |

fitted with a 6 X 20 cm PVC tube wrapped in cheese cloth which served as a 565 cm³ place holder for the planting hole. The cloth prevented materials from falling into the tube. The tube was removed after the compaction process. This prevented undue disturbance of the compacted profile while allowing for a planting hole. The tubes were placed slightly below the plane of the top of the container to prevent vibratory effects during compaction. The containers with the stone–soil blends were then compacted.

To compact the test blends, all containers were blocked pot to pot and covered with a geotextile. The geotextile was covered with a 1.5 inch layer of #2 stone and then compacted with a vibratory plate tamper (Wacker VPG 160K). The fabric and stone deformed into the containers as the media settled, maintaining media/tamper contact for uniform compaction. Compaction consisted of four passes with the plate tamper; care was taken to pass the center of the tamper over all edges of the block for uniformity of compactive effort. The coverings were removed, and the initial excess test blend was replaced into the containers where settlements had occurred. The pots were again covered and four more passes with the tamper were performed. Controls were compacted in four lifts with an impact hammer method instead of a vibratory plate tamper due to the fine nature of the clay loam. During the plant harvest, the final densities and porosities were calculated (Table 2).

On June 9,1993, dormant *Tilia cordata* seedlings with swollen buds were standardized to a single stem of 50 cm. The root systems were standardized to a single root of 15 cm with all laterals and the root tip removed. Planting tubes were slid out of the compacted containers, and lindens were installed with the same shredded, noncompacted clay loam as was used to fill the interstitial voids in the stone–soil mixes. Plants were watered in after planting and placed into a completely randomized experimental block. They were grown on an outdoor gravel pad in Ithaca, NY, kept weed free, and watered as needed until the end of August, 1993.

Plants were forced into an early dormancy after the trees had set terminal bud by placing them into a 6°C cooler on August 31, 1993. After approximately three months of chilling, the plants were placed into a greenhouse in a completely randomized experimental design on December 8, 1993. The plants received 16-hour day lengths using supplemental incandescent lighting. The greenhouse temperatures were maintained at 21°C/15.5°C day/night and plants were watered as needed.

The trees were harvested beginning on March 28, 1994, once they had again set terminal bud. At harvest, the final volume of each test container was calculated by taking the average of four measurements from the top of the container to the soil surface (one from each quadrant of the container), and subtracting the empty volume from the total pot volume. The final weight and moisture content was measured and the final dry density calculated.

The root harvest consisted of a total root excavation and collection. The initial standardized root was removed, and the remaining roots were washed free of soil. The volume of new root growth was measured using water displacement in a graduated cylinder. The roots were viewed as cylinders with a diameter equal to the average root diameter which was estimated to be 1.5 mm yielding an average root radius of 0.75 mm. By taking the water displacement of the roots as the volume of these root cylinders, root lengths were calculated (Table 3) from the following constant relationship: Length (cm) = Volume ($cm^3$) ÷ [pi x $(0.075cm)^2$]. This transformation was done to more effectively communicate root growth by length rather by volume. Since the data were transformed by a constant factor, any treatment differences were not obscured or developed. Plants were harvested following the randomized design. Due to the number of plants and the painstaking nature of the root excavation, the harvest lasted from March 28 to April 29, 1994.

**Engineering behavior of limestone based media.** Initial determination of the engineering properties of the blends was accomplished through the testing of the limestone based medium, which was chosen for its manufactured consistency. A series of limestone media were blended in batches in the same manner as described in the linden study. Blends were based on a 100 kg stone component contribution. Based on the linden study observations, the initial hydrogel tackifier rate was thought to be higher than needed and was therefore reduced to 38 g of hydrogel per 100 kg of stone in the engineering tests.

**Table 3. Response of linden root development by treatment. Overall standard error by treatment = 348.4 cm excepting where single replicates had died (X); in which case the standard error = 381.6 cm.**

| Stone type | Stone to soil ratio | Avg. root length (cm) without hydrogel | with hydrogel |
|---|---|---|---|
| Limestone | 368:1 | 1971 | 3216 |
| | 4.09:1 | 2264 | 1879(X) |
| | 4.60:1 | 2047 | 2839 |
| | 5.26:1 | 1947(X) | 2773(X) |
| High friction | 5.47:1 | 2377 | 2584 |
| | 6.03:1 | 2509 | 1999 |
| | 6.84:1 | 1981 | 3103 |
| | 7.81:1 | 3169 | 2462 |
| Solite ® | 1.46:1 | 2528 | 2726 |
| | 1.70:1 | 2811 | 2084 |
| | 1.78:1 | 2113 | 2226 |
| | 2.09:1 | 2467(X) | 2433 |
| Clay loam | — | 586 (X) | 3640 |

In the linden study, matrix pore volumes were calculated for noncompacted stone. When the matrix was compacted, the resulting matrix pore volumes were reduced. Therefore, the soil that was initially measured to volumetrically fill 70, 80, 90, and 100% of the noncompacted matrix voids would now be found to be compacted at least at the 90 and 100% levels (the two lowest stone to soil ratios for each stone type). After looking at the final compacted stone matrix and the bulk density of the clay loam used in the study, the soil volumes used would overfill the interstitial voids at the stone to soil ratios used unless the soil was compacted. For this reason, the mixes tested in the engineering phase of the study represented stone to soil ratios ranging from 4:1 to 7:1 (Table 4). The mixes also represented a range which would start to define a critical stone to soil ratio and maximize the soil component of the system.

Moisture density relationships were determined following standard Proctor testing methods (ASTM D 698 method D) [2] with the following modifica-

**Table 4. Observed maximum densities and associated moisture contents of limestone blends resulting from standard and increased Proctor type compaction efforts. Porosities are calculated for each density.**

| Stone to soil ratio | Observed maximum dry density from 592.7 kJ/m³ effort | Observed optimum moisture content±1% | Porosity at optimum density from 592.7 kJ/m³ effort | Observed maximum dry density from 1609 kJ/m³ effort | Observed optimum moisture content | Porosity at optimum density from kJ/m³ effort |
|---|---|---|---|---|---|---|
| 4.057:1 | 1990 | 12.2 | 26% | 2030 | 12.1 - 13.0 | 24% |
| 4.997:1 | 1970 | 12.0 | 27% | 2050 | 9.0 - 12.0 | 24% |
| 5.026:1 | 1960 | 11.8 | 26% | 2040 | 8.0 - 12.5 | 23% |
| 6.28:1 | 1920 | 11.0 | 29% | 2030 | 8.5 - 11.5 | 25% |
| 7.085:1 | 1910 | 11.8 | 29% | 2000 | 11.5 - 13.0 | 26% |

tions. No sieving of the materials was done since 15%+ of the material would be retained on the 0.75" sieve and its removal would radically change the tested blend. The 6" mold was chosen to accommodate the large aggregate. A metal rod was thrust 21 times at the edges of the mold in the initial lift to prevent bridging of the stones against the base of the mold. This bridging would have created inordinately large voids at the base of the compacted profile yielding inaccurate dry density calculations. Screeding the material level with the top of the mold for accurate volume calculation required the removal of stone that extended into the mold. Upon removal, smaller stone particles and soil were replaced into the mold in an approximation of the stone to soil ratio of the tested material. This material was packed by hand and pressure applied with the screeding bar to reduce the potential difference in compactive effort in those replaced areas.

The standard compactive effort of 12,375 ftlb/ft³ (592.7 kJ/m³) in three lifts was applied in the manner described earlier. Moisture density curves were based on seven resultant density test observations at increasing moisture levels. All test materials were allowed to sit for 24 hours in closed containers to allow for equilibration of moisture content since water was added to generate each increased moisture content. Specific gravity values for each blend were calculated from the specific gravity of each ingredient (listed in Table 1) and the stone to soil ratio for each blend. The specific gravity for each blend was used to calculate porosity and void ratios for that material at various oven-dry densities.

A second set of compactions using a 10 lb (4.54 kg) hammer, 18" (457 mm) drop, 3 lifts, and 56 blows per lift (ASTM D1557 method D in only three lifts) were also completed. This resulted in a 33,592 ftlb/ft³ (1609 kJ/m³) compaction effort. Determination of the expected standard proctor optimum density for all stone–soil blends was calculated by first compacting each stone type with the standard 592.7 kJ/m³ effort and determining its density as an average of five tests. By adding to this the dry weight of noncompacted clay loam soil for each stone to soil ratio, a predicted optimum dry density for each mix was calculated.

Variability of moisture content in each test was estimated to be ±1% due to the stoniness and the rapid drainage capacity of these blends. Variation in dry density was assigned at ±7.5 kg/m³ calculated from the specific gravity of the blend and the size of the mold.

California Bearing Ratios were determined on test blends with limestone to soil ratios of 4.057:1 and 5.026:1 to see how they would sustain loading and to judge their efficacy as potential pavement bases. CBR testing was conducted on soaked samples following the ASTM 1883 protocol [2]. Materials from limestone stone to soil ratio 4.057:1 were compacted with a 1609 kJ/m³ effort (Table 5). Materials for the limestone stone to soil ratio 5.026:1 were compacted using the standard 592.7 kJ/m³ and 1609 kJ/m³ efforts. Piston seat weight

**Table 5. CBR testing results from two limestone test media. All tests were subjected to a 96 hour saturation period by submersion.**

| Stone to soil ratio | Comparative effort ($kJ/m^3$) | Moisture content (%) during compaction | Resultant dry density ($kg/m^3$) | CBR at 2.5 mm penetration | CBR at 12.5 mm penetration | Post CBR test moisture content (%) | Surcharge used during saturation period (kg) |
|---|---|---|---|---|---|---|---|
| **4.057:1** | 592.7 | 9.5 | 1961 | 48 | 57.2 | 11.8 | 6.936 |
| | 592.7 | 9.5 | 2068 | 76 | 73.5 | 11.2 | 5.71 |
| | 592.7 | 9.5 | 1946 | 49 | 45.9 | 11.5 | 5.749 |
| | 1609 | 9.9 | 2044 | 65 | 113.3 | 9.8 | 5.726 |
| | 1609 | 9.9 | 2081 | 99 | 93.9 | 10.5 | 5.748 |
| **5.026:1** | 1609 | 9.2 | 2042 | 65.1 | 64.1 | 11.2 | 5.715 |
| | 1609 | 9.2 | 2015 | 101.5 | 105.3 | 9.4 | 5.748 |
| | 1609 | 9.2 | 2025 | 125.3 | 78.6 | 11.3 | 5.737 |
| | 1609 | 9.2 | 2055 | 96.5 | 93 | 10.8 | 5.731 |
| | 1609 | 9.2 | 2005 | 95.3 | 82.4 | 9.9 | 6.936 |
| | 1609 | 9.2 | 1983 | 79 | 80 | 10.6 | 6.943 |

on all specimens was 6.75 lbs. All samples were soaked by submersion for 96 hours and drained for 15 minutes prior to testing. During the soaking period, all samples experienced a metal surcharge of 5715–6943 g to simulate a pavement layer over the test material during the saturation period (Table 5). The penetration rate of the piston was slowed to a uniform 0.025 inches/minute and readings were taken as each stone breakage registered and at the ASTM standard recording depths. The curves were generated using these points and then corrected as per ASTM 1883 [2,26]. The resultant curve could consequently be inflated, but would more accurately reflect each material's behavior in comparison to only the predetermined depth readings and would be a function of the stone's inherent strength.

## Results and Discussion

**Linden study.** Roots in the compacted nonhydrogel controls were observed only in the initial noncompacted planting tube area except in one replication. In the one replicate where roots did penetrate the soil, roots followed the interface between two lift compaction zones and grew toward the side of the container but did not reach it.

In all other stone and soil test media the roots were observed to reach the bottoms and the sides of the containers throughout the entire profile. Occasionally, roots were seen to grow around zones of poor aeration where uneven mixing left high concentrations of hydrogel. This problem was observed in 4 replicates of the mix containing the highest proportion of soil (4.09 parts limestone to 1 part soil) with hydrogel. Mycorrhizae were observed in nearly all test containers, with exceptions occurring randomly across the entire range of test media.

Root growth was impeded in the control without hydrogel compared to all other blends and the addition of hydrogel to the control increased root penetration by 621% over the nonhydrogel control (Table 3). The bulk density of the clay loam with hydrogel was 1.25 as opposed to 1.38 without hydrogel (Table 2). This could be attributed to swelling of the hydrogel in the soil separating the soil aggregates reducing the dry density of the soil. This would also create relatively large pores which would allow for vigorous root growth. There were no significant differences between the stone types or the stone to soil ratios. There was no significant effect on root penetration caused by the use of the hydrogel, type of stone, or stone to soil ratio. No interactions were found. All treatments significantly improved root growth when compared to the control ($p<.001$). Root length in the stone–soil blends ranged from 1879 cm to 3216 cm, an improvement of 320–548% over the soil control (Table 3).

The overall low standard error of observed density indicated that compaction variability between replicates in the linden study was low. Standard errors of density ranged from 25 to 27 $kg/m^3$ in systems with densities from 1090 to 1852 $kg/m^3$ (Table 2). Since there were differences in specific gravities among stone types, it would not be appropriate to compare groups by observed densities alone. More revealing was the effect of hydrogel on density for each treatment and as a percentage of optimum density for each treatment.

As the stone to soil ratio increased, the difference in density between each mix with and without hydrogel decreased (Table 2). This may indicate that in the lower stone to soil ratios, the water absorbed by the hydrogel held the matrix stone apart during compaction. By comparing the dry densities of the linden study mixes and factoring in the particle density of the solids, we calculated the porosity of the mixture (Table 6). The hydrogel rate used in the linden study was approximately 150 g hydrogel/100 kg stone. If the material absorbed 200 times its weight in water, it would have been able to hold enough water to cause compaction of the clay loam if the blends had been compacted to Standard Proctor Density. As the stone to soil ratio increased, empty pore spaces in the matrix would have to increase. The increased empty pore volume would allow space for the hydrogel to swell without displacing the matrix, resulting in less of a difference between hydrogel and nonhydrogel treatments of the same stone/soil mix.

**Table 6. Porosity of non-compacted loam used to create the blends = 57.1%. Comparative porosities of stone/soil mixes if compacted to standard Proctor optimum density. Porosity of interstitial soil also shown, from dividing volume of voids by the volume of soil solids. The stone is treated as inert space and is ignored in the calculation.**

| Stone | Stone to soil ratio | Porosity blend at standard Proctor optimum density (%) | %soil solids in compacted profile by volume | Porosity soil within the stone matrix (%) |
|---|---|---|---|---|
| **Limestone** | | | | |
| | 3.68:1 | 25.5 | 19.3 | 56.9 |
| | 4.09:1 | 26 | 17.7 | 59.5 |
| | 4.6:1 | 26.4 | 16 | 62.3 |
| | 5.26:1 | 27 | 14.3 | 65.5 |
| **High friction** | | | | |
| | 5.47:1 | 21.9 | 14.4 | 60.4 |
| | 6.03:1 | 23.1 | 13.1 | 63.8 |
| | 6.84:1 | 24.4 | 11.6 | 67.7 |
| | 7.81:1 | 25.6 | 10.3 | 71.4 |

All of the blends were classified by the Unified Classification System. The blends were characterized by stone type and particle size distribution showing their gap–graded nature (Figure 5). The limestone blends ranged from gravel–silt mixture/clayey–gravel (GM–GC) to a poorly graded gravel/gravel–silt mixture/clayey–gravel (GP–GM–GC). The high friction aggregate mixes ranged from a poorly graded gravel/gravel–silt mixture/clayey–gravel (GP–GM–GC) to a poorly graded gravel (GP). The Solite blends all fell into the gravel–silt mixture/clayey–gravel (GM– GC) category (Figure 5).

It would appear that the non–Solite® stone–soil blends would serve as an excellent subbase, and a good base at the higher stone to soil ratios (Figure 4). Also, the non–Solite® blends normally would exhibit only a slight susceptibility to frost action [4,5,28]. Although Solite® blends compared poorly with the non–Solite® stone blends, care should be taken before discounting the Solite® since the classification is based on the weight of the particles and Solite®, being a heat expanded slate, is very light per unit particle size when compared to the clay loam due to entrapped air voids within the aggregate. The Unified Classification may not be a valid predictor of performance in this unusual case. Observations of root growth indicated that Solite® behaved similarly to the other stone types (Table 3).

Comparison of the porosity of the uncompacted clay loam (57.1%) with the calculated porosities within the interstitial spaces of the stone matrices (56.9%–71.4%), showed that the soil within the stone matrix had equal to or greater porosity than the uncompacted soil (Table 6). The porosity of the uncompacted loam was in fact between 10 and 30% less than the soil within the stone matrix [9]. This would explain why root growth was unimpeded in all of the stone/soil blends as compared to the compacted soil without hydrogel control; despite the higher densities of the stone/soil blends. This is good evidence for using the porosity of the soil within the stone matrix spaces and not the porosity of the total stone and soil system as the critical measurement. If root growth was impeded with 45.4% porosity in the compacted clay without hydrogel, then it would seem surely to be impeded with the 22 – 27% overall porosity in the stone/soil blends, (Table 6) yet root growth increased a minimum of 320% over the compacted soil control (Table 3).

Since there were obvious increases in root growth over the control in all treatments, there was reason to believe that this type of system can be used to successfully sustain street tree root growth. This system will allow root penetration and normal short term growth over a wide range of stone to soil ratios when compacted to 80% standard Proctor Optimum Density. Studies at higher densities are underway.

**Engineering studies.** The soil mix density was seen to consistently increase as expected with the increased 1609 kJ/m$^3$ compactive effort and with one exception, with increased amounts of soil in the stone matrix (Table 4). The exception involved limestone 4.057:1 where, at the 1609 kJ/m$^3$ effort, the density observed actually dropped over 2.5 times beyond the assigned margin of error of 7.5 kg/m$^3$. This was taken to indicate that a critical stone to soil ratio had been crossed, and the soil portion of the blend had possibly impacted the formation of the stone matrix.

A minimum CBR value of 40 was considered satisfactory, and all tested blends showed an adequate CBR rating (Table 5). All samples would have been an acceptable base under saturated conditions (the worst case scenario) provided that the pavement was thick enough to withstand the projected maximum load of the sidewalk.

In the remaining ten samples, the CBR values covered a range of 60 units (a very wide range) over densities that varied from 1946 to 2081 kg/m$^3$ (a very narrow range). This difference in range could be caused by uneven stone breakage during the test. The surface area of contact of the piston and the depth of the penetration affect the measured CBR as does the placement of the piston in relation to the stones beneath the piston and the location and timing of stone breakage. Resistance to load would increase to a point of stone failure and plummet to a lower resistance until the next stone was encountered. This is not surprising due to the open nature of the matrix and the ability of shattering stones to quickly nest into surrounding voids. For this reason, results should focus on an acceptable range of CBR values in relation to density in this type of mix rather than a single measurement.

It does appear from the initial tests, that the materials used in this study would be considered acceptable for use as a subbase or as a base under light traffic pavement structures. The linden study has demonstrated that these same materials have the potential to allow for vigorous root growth. Normally, materials in these classes would be expected to possess a low frost-heave potential [4]. In the blends developed in this study, frost heave potential would likely be a function of the amount of hydrogel in the blend since it normally absorbs up to 300 times its own weight in water. However, it would be reasonable to believe that the material would be less frost sensitive than current materials in use if pore space existed in large enough voids to allow for the expansion of ice lenses without disturbing the matrix. The rate of hydrogel used in the system now becomes an influential factor, and testing must be done to further define this rate. However, at the rate of 38 grams of hydrogel per 100 kg of stone, fully hydrated gel would occupy only 1% or less of the matrix pores.

### The Critical Stone to Soil Ratio

If one accepts the assumption that both the stone and the water in such systems are incompressible, then there is a critical stone to soil ratio similar to the threshold proportion of sand discussed by Spomer for landscape soils [24]. Below this critical ratio, the excessive soil in the system would either be compacted, impact the formation of the stone matrix, or affect the engineering properties of the total system. By having more soil in the system than could be accommodated by the pores in the compacted stone matrix, the soil would be compacted. In this case, the stones would “float” in the compacted soil and not come into contact with other stones thus preventing the bridging of the stones which form the load–bearing stone matrix. In this situation, the engineering behavior would be that of the soil and not of the stone, and the soil would be compacted to the same problematic levels in order to bear loading.

Critical dry weight stone to soil ratios are different from that mathematically expected and will be unique for each stone type and shape and for each soil used. In practice, as the stone and soil were mixed and compacted, the soil would be unavoidably compacted to some extent. This would happen even when the stone matrix pores were only partially filled with soil. With the introduction of hydrogel into the system, additional incompressible water would act as another additional compactive force on the soil within the system. As the fine materials in soil were added to the stone matrix, the matrix would form differently. Since this is a dry weight ratio, the particle density of the stone and soil used will have a direct effect on the critical ratio. Highly angular stone will have a different compacted matrix porosity when compared to a rounded stone, and will accept additional soil volumes. Since the critical stone to soil ratio will be affected by the stone type and by soil type, a generalized critical stone to soil ratio or equation is yet to be thoroughly identified.

A way to estimate this critical stone to soil ratio would be to chart the observed optimum density of a mix in relation to its calculated optimum density. The calculated optimum density is the compacted stone matrix density with the ratio weight of soil added. For this initial calculated density, the assumption has been made that this addition of fine material will not substantially change the final compacted stone matrix. Since the ratio of stone to soil could be considered constant regardless of the density, a change in the relationship between the calculated and observed optimal density would indicate a change in the stone/soil system. A ratio of calculated to observed optimal densities greater than 1.0 would indicate soil compaction had occurred, impacting the final stone matrix. Since the overall difference in the soil component over the entire range of tested materials is relatively small, this type of shift in density behavior is likely a change in the stone matrix. Below the 5:1 stone to soil ratio there appears to be a shift in density which would indicate that soil compaction within the stone voids had occurred in the crushed limestone experimental system (Figure 6).

## Summary

It is apparent that we can grow plant materials in a load bearing pavement base. The linden study showed vigorous and healthy root growth in compacted profiles in excess of 1700 kg/m$^3$ bulk density while roots in controls of compacted clay loam to 1377 kg/m$^3$ were severely impeded. Initial engineering tests of a crushed limestone media indicated that the blends would function well as pavement bases if compacted to a density of 2000 kg/m$^3$. A blends's strength is a function of the strength of stone if the stone to soil ratio is not lower than the critical ratio which would occur with the addition of excessive soil. For the one system we have tested, the critical ratio was defined as 5 parts crushed #2 limestone to 1 part clay loam soil by dry weight. Extension of this system to various stone and soil types needs to be studied as well as rates and types of hydrogel tackifiers.

Currently plant materials are being grown and studied in stone/soil mixes compacted to optimum densities. Water management, nutrient availabil-

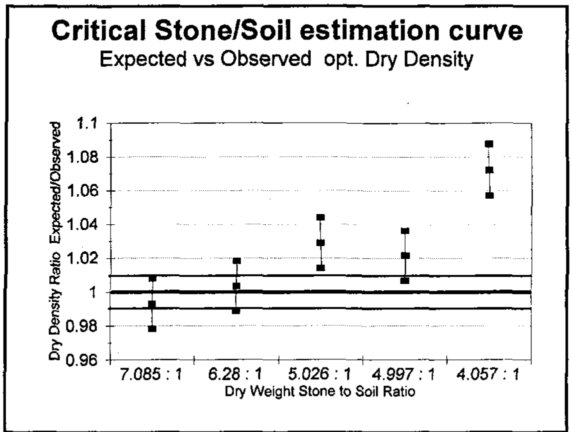



**Figure 6. Graphic estimation of a likely critical ratio for limestone mixes. A ratio above one would indicate the critical stone to soil ratio had been crossed and compaction of the interstitial soil had likely experienced compaction. Error bars represent the range of ratio values due to the assigned acceptable error of each of the measurements (kg/m$^3$)**

ity, frost-heave susceptibility, as well as root and shoot growth studies will be incorporated into these and future tests. Field installations and further laboratory testing are underway to further quantify and generate a common specification for stone and soil mixtures for commercial use.

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## Literature Cited

1. American Association of Highway and Transportation Officials. 1986. AASHTO Guide For Design of Pavement Structures. AASHTO. Washington. 297 pp.
2. American Society of Testing and Materials. 1993. Section 4 Construction Vol. 4.08 Soil and Rock; Dimensional Stone; Geosynthetics. Annual Book of ASTM Standards. ASTM ed. American Society for Testing and Materials. Philadelphia. 1470 pp.
3. Barley, K. P. *Influence of soil strength on growth of roots.* 1963. Soil Sci. **96**(1): 175-180.
4. Berg, R. and T. Johnson. 1983. Revised Procedure for Pavement Design under Seasonal Frost Conditions. Special Report 83-27 of the US Army Corps of Engineers Cold Regions Research & Engineering Laboratory ed. Cold Regions Research & Engineering Laboratory US Army Corps of Engineers. Hanover. 129 pp.
5. Chou, Y. T. 1977. Engineering Behavior of Pavement Materials: State Of The Art. Technical Report S-77-9 of the US Army Engineer Waterways Experiment Station ed. United States Government Printing Office. Washington.409 pp.
6. Craul, P. J. 1992. Urban Soil in Landscape Design. John Wiley & Sons, Inc. New York. 396 pp.
7. Das, B. M. 1985. Principles of Geotechnical Engineering. PWS Engineering. Boston. 571 pp.
8. Eavis, B. W. and D. Payne. 1968. Soil Physical Conditions and Root Growth, pp 315–338. **In** Root Growth. Whittington ed. Butterworths. London.
9. Grabosky, J. 1995. Identification and testing of load bearing media to accommodate sustained root growth in urban street tree plantings. M.S. Thesis, Cornell Univ.
10. Heilman, P. 1981. *Root penetration of Douglas-fir seedlings into compacted soil.* Forest Sci. 27(4): 660-666.
11. Holtz, R. D. and W. D. Kovacs. 1981. An Introduction to Geotechnical Engineering. Prentice-Hall Inc. Englewood Cliffs NJ. 733 pp.
12. Johnson, A. W. and J. R. Sallberg. 1960. Factors that influence field compaction of soils. Bulletin 272 of the National Academy of Sciences - National Research Council,. Wahington. 206 pp.
13. Lindsey, P. and N. Bassuk. 1992. *Redesigning the urban forest from the ground below: A new approach to specifying adequate soil volumes for street trees.* J. Arboric. 16: 25-39.
14. Means, R. E. and J. V. Parcher. 1963. Physical Properties of Soils. Charles E. Merrill Books Inc. Columbus OH. 464 pp.
15. Patterson, J. C. 1977. *Soil compaction - effects on urban vegetation.* J. Arboric. 3(9): 161-166.
16. Patterson, J. C., J. J. Murray and J. R. Short. 1980. The impact of urban soils on vegetation, pp 33–56. **In** Proceedings of the third conference of the Metropolitan Tree Improvement Alliance (METRIA).
17. Perry, T. O. 1980. The size, design and management of planting sites required for healthy tree trowth, pp 1–14. **In** The Proceedings of the third conference of the Metropolitan Tree Improvement Alliance (METRIA).
18. Perry, T. O. 1982. *The ecology of tree roots and the practical significance thereof.* J. Arboric. 8(8): 197-211.
19. Proctor, R. R. 1933. *Description of field and laboratory methods.* Eng. News-Rec. 111(10): 286-289.
20. Proctor, R. R. 1933. *Field and laboratory verification of soil suitability.* Eng. News-Rec. 111(12): 348-351.
21. Proctor, R. R. 1933. *Fundamental principles of soil compaction.* Eng. News-Rec. 111(9): 245-248.
22. Proctor, R. R. 1933. *New principles applied to actual dam-building.* Eng. News-Rec. 111(13): 372-376.
23. Scott, C. R. 1980. An Introduction to Soil Mechanics and Foundations. Applied Science Publishers Ltd. London. 406 pp.
24. Spomer, L. A. 1983. *Physical amendment of landscape soils.* J. Environ. Hort. 1(3): 77–80.
25. Taylor, H. M. 1971. Root behavior as affected by soil structure and strength, pp 271–292. **In** The Plant Root and Its Environment. Carson ed. University Press of Virginia. Charlettsville.
26. The Asphalt Institute. 1978. Soils manual for the design of asphalt pavement structures. Manual Series No. 10. Institute ed. The Asphalt Institute. College Park MD. 238 pp.
27. US Army Engineer Waterways Experiment Station. 1959. Developing A Set of CBR Design Curves. United States Government Printing Office. Washington. 24 pp.
28. US Army Engineers Waterways Experiment Station. The Unified Soil Classification System and Appendix. 1960 United States Government Printing Office. Washington. 30pp A11.
29. US Department of the Interior and Bureau of Reclamation. "Earth Manual." 1974 United States Government Printing Office. Washington. 810 pp.
30. USDA Soil Conservation Service. Soil taxonomy: A basic system of soil classification for making and interpreting soil surveys. No. 436(Dec. 1975): 1975. 754pp.

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**Zusammenfassung.** Die Bodenverdichtungen, die zur Sicherung und Stabilisierung von Strassenbau und Pflasterarbeiten notwendig sind, stehen im Konflikt mit dem *Anspruch auf ausreichendem Wurzelraum, um gesundes* Baumwachstum zu unterstützen. Wir haben ein ausdauerndes Bodenmedium herausgefunden, das die ingenieurtechnischen Ansprüche an Traglasten erfüllt und darüberhinaus rasches Wurzelwachstum und Wurzelausdehnung ermöglicht. Das wird gewährleistet durch ein grobporiges Steingranulat und untergemischtem Boden mit der Unterstützung von einer klebenden hydrogenen Trägersubstanz. Die Eingangsstudien, bei denen drei Steintyen und verschiedene Granulat : Boden - Verhältnisse getestet wurden, zeigten, daß das verdichtete Granulat : Boden - Gemisch das Wurzelwachstum um 320% gegenüber herkömmlichen verdichtetem Boden steigerte. Das vorgeschlagene System kann sicher Lasten tragen, wie es *demonstriert werden konnte durch die Überschreitung* des kalifornischen Traglastenverhältnisses um 40. Das Mischungsverhältnis wurde kritisch untersucht, um Richtlinien für die praktische Anwendung zu entwickeln.