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(Consultant)

M E M O R A N D U M

TO: Allan Frank, P.E.
Division of Structural Design

FROM: William Broyles, P.E.
Geotechnical Branch Manager

BY: Bart Asher, P.E., P.L.S. BA
Geotechnical Branch

DATE: April 25, 2007

SUBJECT: **Jefferson County**
1200 056
Mars # 6554101D
Ramp 42 3RD Street to I-65 SB (S0180, Bridge B3RD-1)
Ohio River Bridges Project
Kennedy Interchange Reconstruction – Section 1
Item No. 5-118.18 & .19

The geotechnical engineering report for this structure has been completed by Barr & Prevost. We have reviewed and concur with the recommendations as presented in this report.

A copy of the report is attached. If you have any questions, please contact this office at 502-564-2374.

Attachment:

cc:	J. Callihan		
	R. Harris		
	N. Stroop		
	A. Calvin		
	B. Greene		
	B. Bryant		
	Glen Kelly	(KTA-QK4)	(w/o attachment)
	Gary Simmons	(Barr & Prevost)	(w/o attachment)
	Stuart Edwards	(Barr & Prevost)	(w/o attachment)

FINAL REPORT
GEOTECHNICAL INVESTIGATION
KENNEDY INTERCHANGE RECONSTRUCTION
RAMP 42 3RD STREET to I-65 SB
(S0180, BRIDGE B3RD-1)
JEFFERSON COUNTY, KENTUCKY

For:

Kentucky Transportation Associates
815 West Market Street, Suite 300
Louisville, KY 40202

Submitted by:

BARR & PREVOST
ENGINEERING | TESTING | SURVEYING

9420 Towne Square Avenue, Suite 22
Cincinnati, OH 45242
Voice: 513.936.9400
Fax: 513.936.8400

April 18, 2007

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(S0180, BRIDGE B3RD-1)
JEFFERSON COUNTY, KENTUCKY
FOR
KENTUCKY TRANSPORTATION ASSOCIATES

1. LOCATION AND DESCRIPTION

This report presents the results of our geotechnical investigation for part of the proposed new ramp 42 carrying traffic from the 3RD Street on-ramp to Southbound (SB) I-65, north of downtown Louisville, Kentucky. This structure is part of the Kennedy Interchange Reconstruction, in turn a component of the Louisville-Southern Indiana Ohio River Bridges Project. The project area is north of the Louisville central business district in Jefferson County. The bridge begins at the north end of 3RD Street and ends at the beginning of Bridge B3RD-8 (Structure S0660). A Site Location Map is presented as Exhibit 1.

Working documents and drawings germane to the Kennedy Interchange Reconstruction are stored on ProjectWise, an internet-based data management system, utilized by Kentucky Transportation Associates' (KTA) team members and subcontractors. Subsequent reference to this system will take the form "ProjectWise 061014" referencing a document available on October 14, 2006. The bridge is being designed using the Load and Resistance Factor Design (LRFD) method as set forth in the American Association of State Highway and Transportation Officials (AASHTO) Publication "LRFD Bridge Design Specifications 4th Edition". Where this document is referenced subsequently, the term ?LRFD BDS' will be used.

According to the plans prepared by WMB, (ProjectWise 070216), the four-span bridge has a total length of 578.09 feet. The bridge section consists of an 8-foot inside shoulder, 15-foot lane width and a 6-foot outside shoulder. The initial part of Ramp 42 is built on an embankment that increases in height from ground level to about 14 feet at Abutment 1 of B3RD-1. The bridge then rises at a 5 percent (%) grade up to its junction with the Great Lawn Structures at Pier 4. The proposed start/finish substructure centerlines are shown in Table 1. The bridge plan is presented in Exhibit 2.

Table 1: Proposed Substructures Stationing[1]

Structure	Substructure	Centerline Station
B3RD-1	Abutment 1	423+81.92
B3RD-1	Pier 1	424+97.25
B3RD-1	Pier 2	426+09.58
B3RD-1	Pier 3	427+79.25
B3RD-1	Pier 4	429+59.92

(1) Preferred Layout, ProjectWise 070316.

The purpose of this investigation is to characterize the subsurface conditions of the site, perform geotechnical engineering analyses, and provide recommendations for the design and construction of the foundations of the bridge piers and abutment.

2. SITE TOPOGRAPHY AND GEOLOGIC CONDITIONS

2.1 General

The proposed ramp alignment crosses both West River Road and Bingham Way. The existing terrain is relatively flat varying in elevation from about 438 – 440 feet mean sea level (msl) through Pier 3 and about 444 feet msl under the final span.

Buried utilities have been mapped in the area as shown in ProjectWise 070216; those relevant to foundation construction appear to include:

- 12-inch combined sewer directly beneath Abutment 1
- 15-inch sanitary sewer directly beneath Pier 2
- 12-inch sanitary sewer beneath possible pile cap for Pier 3
- 12-inch sanitary sewer immediately adjacent and north of possible Pier 4 pile cap

Each of these will be potentially impacted by the installation of deep foundations.

2.2 Geology

The interchange system will be constructed in close proximity to the Ohio River, and generally within its left bank flood plain. Subsurface conditions are, therefore, dominated by the morphology of the river valley and more recent depositional history.

In the Louisville area, the Ohio River flows through a broad, relatively flat valley, carved into bedrock during post-glacial periods. Bedrock typically consists of Devonian and Silurian-age limestone, dolomite and shale; the valley sides southeast of the site area are composed of Sellersburg and Jeffersonville limestone, the base of which occurs at about elevation 480-490 feet msl. The geologic sequence below this is reported as follows:

Table 2: Geologic Sequence – Louisville, Kentucky

Formation	Rock Type	Thickness (ft.)	Base Elevation[1] (Ft. – msl)
Jefferson	limestone	18-30	480-490
Louisville	limestone	45-75	405-445
Waldren	shale	9-13	392-436
Laurel	dolomite	50	346-386
Osgood	shale/dolomite	17	329-369
Brassfield	dolomite	10-30	200-359
Drake	dolomite, limestone, shale and mud stone	55-150	149-304

(1) Calculated based on mapped contact elevation at base of Jefferson Limestone.

Bedrock in the floor of the buried valley is recorded as being between 334 feet and 337 feet at more than a dozen well sites in the 2-mile stretch of the aquifer upstream from the current I-71/I-64 interchange (Unthank & Nelson, 2006). It is likely that the valley floor continues downstream, with some variation, at about that elevation into the area covered by this investigation. Based on the geological information presented above, the valley floor could be in the Osgood, Brassfield or Drake formations. The composition is therefore uncertain, as these tend to be composed of a variety of rock types, although dominantly of dolomite.

Bedrock is masked in the broad flood plain by Quaternary sediments that are frequently in excess of 100-feet thick. These consist of glacial outwash and river alluvium. Locally, man-made fill is present. The alluvium is generally less than 35 feet in thickness and may consist of clay, sand or gravel. Outwash deposits typically comprise sand and gravel, coarsening with depth and generally extending to bedrock. A geologic map is presented as Exhibit 3.

The subsurface investigation results are consistent with this geologic setting. The typical observed stratigraphy consists of man-made fill (~10 feet) overlying alluvial clay (~20 feet) followed by outwash deposits (sand and gravel mixtures) to the total depth explored (80 – 116 feet).

Groundwater conditions in the flood plain are influenced heavily by the stage of the Ohio River. In the extreme, flood levels up to elevation 450 feet (100-year flood) may be experienced, but lesser floods can be expected annually that are capable of inundating the project area. The river stage is regulated and for much of the year the normal stage for McAlpine Upper Pool is elevation 420 feet. Groundwater levels near the river may be expected at a similar or slightly higher elevation.

2.3 Seismicity

The seismic acceleration coefficient (A) for the site is derived from contour maps prepared by the U.S. Geological Survey (USGS) (LRFD BDS Figures 3.10.2-1-3) for the design earthquake. The Louisville area coefficient is 6% (corresponding to an acceleration of 0.06 times gravitational acceleration). This places the structure in Seismic Zone 1 ($A \leq 0.09$). There is a 90% probability that this acceleration will not be exceeded during a 50-year period. The return frequency for such an event is about 475 years.

Design specifications for seismic design are currently under review at AASHTO and are expected to undergo major changes, perhaps as soon as 2007 (Imbsen, 2006). The Designer should be aware of the status of these changes as the design moves forward and request ongoing assistance from a geotechnical engineer, as appropriate.

3. FIELD INVESTIGATION PROGRAM

Drilling and sampling were performed by Fuller, Mossbarger, Scott and May Engineers, Inc. (FMSM) of Lexington, Kentucky between May 22 and 31, 2006. The subsurface exploration consisted of three structure borings, designated as Borings 4B-248, 4B-249, and 4B-250. Boring 4W-270 was drilled for an adjacent structure and serves to increase the level of detail for both. Borings 4B-248 and 4B-249 were drilled to a depth of 80 feet, while Boring 4B-250 was terminated at a depth of 116.5 feet bgs. Boring 4W-270 was drilled to a depth of 80 feet. The borings were marked in the field by a Qk4 survey crew in accordance with an approved plan.

As-drilled boring locations are shown on Exhibit 2, and summaries of the drilling information are presented in Table 3. The Coordinate Data Submission Form, provided by the Kentucky Transportation Cabinet (KYTC) Geotechnical Branch, is completed and included in Appendix A. The logs of three borings drilled in 1962 for construction of the original interchange (Holes #DD, #E, #EE) were consulted for general stratigraphic correlation.

Table 3: Boring Summary for the Recommended Structure

Sub-Structure	Boring Number	Boring Location[1] (Station, Offset)	Surface Elevation (NGDV-Feet)	Bottom of Hole Elevation (Feet)
Abutment 1	4W-270	335+17, 133 RT	445.05	365.05
Abutment 1	4B-248	336+93, 136 RT	443.69	363.69
Pier 1	4B-248	336+93, 136 RT	443.69	363.69
Pier 2	4B-249	338+75, 136 RT	440.58	360.58
Pier 3	4B-249	338+75, 136 RT	440.58	360.58
Pier 4	4B-250	341+95, 101 RT	444.07	327.57

(1) Feet along I-64 Centerline.

The borings were drilled by FMSM using either an all-terrain-vehicle-mounted drill rig, Model CME 55, or a truck-mounted CME 85 equipped with 3.25-inch hollow stem augers. The first 6.5 feet of Boring 4W-270 was drilled by vacuum extraction to address concern for sensitive utilities in the area. Water was used to reduce the amount of “blow-back” of the granular materials once the borings advanced below the surficial clay.

Soil sampling was performed using an auto hammer according to the AASHTO T-206 “Standard Method for Penetration Test and Split Barrel Sampling of Soils.” FMSM indicated that the auto hammer efficiency of this hammer system is 80 percent. Undisturbed samples were obtained from the cohesive soil layers according to AASHTO T-207 “Standard Method for Thin-Walled Tube Sampling of Soils.” The soil samples were obtained at 5-foot intervals starting 2 feet below the existing ground surface. Upon completion, borings were backfilled with soil cuttings.

An engineer monitored the drilling activities and maintained field boring logs. The field logs included results of Standard Penetration Tests (SPT) recorded as blows per 6 inches. These data were used to determine the values shown on the Subsurface Data Sheet (Exhibit 4) as N values representing blows per 12 inches. In the field, the soils were classified according to American Society for Testing and Materials (ASTM) D-2488 “Description and Identification of Soils: Visual – Manual Procedure”.

All split-spoon samples collected in the field were placed in sealed, glass jars and transported to Wang Engineering Inc. (WEI’s) geotechnical laboratory in Lombard, Illinois. At the time the field investigation was conducted, WEI was the responsible geotechnical engineering firm for this structure. All field visual classifications were reviewed in the laboratory.

4. LABORATORY TESTING AND RESULTS

Selected soil specimens recovered from split-spoon and Shelby tube samples were tested for natural moisture content, particle size distribution, Atterberg limits, and unconfined compressive strength after the KYTC approved the laboratory test request. One-dimensional consolidation, and consolidated-undrained triaxial resistance were measured using some of the Boring 4B-248 samples primarily to support analysis of the Abutment 1 foundation. The laboratory tests were performed according to applicable AASHTO or Kentucky Methods.

The test results are shown on the Subsurface Data Sheet (Exhibit 4), and the results of consolidation and triaxial tests are presented in Appendix B. Soils were classified using indices based on ASTM D-2487 "Classification of Soils for Engineering Purposes (Unified Soil Classification System)" and AASHTO M-145 "Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes." The test results were used to establish material properties, and in subsequent engineering analyses to evaluate foundation design and construction alternatives.

4.1 Undisturbed (Shelby) Tube Samples

Undisturbed (Shelby) tube samples of the clay layers were obtained from all borings. The samples were extruded from the tubes, visually described, and trimmed into 6-inch specimens. Unit weight (dry and moist) and natural moisture contents were determined for each specimen. Selected specimens were subjected to engineering classification, unconfined compressive strength, one-dimensional consolidation and consolidated-undrained triaxial tests.

4.1.1 Engineering Classification Testing

Classification testing was performed on selected Shelby tube samples. In most cases, one soil classification test was completed per Shelby tube. Soils from the Shelby tube samples were classified mostly as clay (CL), according to the Unified Soil Classification System, and as A-6(15) and A-6(16), based on the AASHTO classification system.

4.1.2 Unconfined Compressive Strength Tests

The results of unconfined compressive strength tests are presented on the Subsurface Data Sheet (Exhibit 4) and are summarized in Table 4. The undrained shear strength (commonly referred as "undrained cohesion") is one-half of the unconfined compressive strength.

Table 4: Unconfined Compressive Strength Test Results

Boring Number	Location	Sample Interval Depth (ft)	Dry Unit Weight (pcf)	Moist Unit Weight (pcf)	Moisture Content (%)	Unconfined Compressive Strength (tsf)
4B-248	336+93, 136 RT	30 – 32	104.46	128.91	23.41	0.76
4B-249	338+75, 136 RT	20 – 22	103.05	126.64	22.89	1.29
		25 – 27	97.74	121.49	24.29	0.23
4B-250	341+95, 101 RT	15 – 17	96.82	123.92	27.99	0.81
		20 – 22	98.89	125.53	26.94	0.65
		25 – 27	101.49	125.53	23.69	1.84
		30 – 32	127.26	102.10	24.65	0.52

4.1.3 One-Dimensional Consolidation Tests

Two one-dimensional consolidation tests were performed on clay samples from 20 and 30 feet below the ground surface (bgs). The results are summarized in Table 5, as follows:

Table 5: One-Dimensional Consolidation Test Results

Boring/Sample Number	Depth (bgs) (ft)	C_c[(1)]	OCR[(2)]	C_V[(3)] (ft^2/day)
4B-248 #4	20-22	0.189	1.0	0.15
4W-270 #7	30-32	0.298	1.0	0.10

(1) C_c-field corrected compression index
(2) OCR-over consolidation ratio
(3) C_V-consolidation coefficient

Laboratory data are provided in Appendix B.

4.1.4 CU Triaxial Test

A consolidated-undrained triaxial test was performed on a sample of clay from Boring 4B-248 at 20-27 feet bgs. The effective friction angle (ϕ') was 32.1 degrees and the effective cohesion (c') was zero. Laboratory data are provided in Appendix B.

4.2 Standard Penetration Test Samples

Standard Penetration Tests (SPT) and split-barrel (commonly known as split-spoon) sampling of soils were performed in granular materials or where drilling resulted in poor Shelby tube recovery. SPT N-values ranged from 0 (push) to high values typically in the mid-thirties [blows per foot (bpf)]. The SPT N-values and laboratory test results, which include natural moisture contents, fine (silt and clay) contents, and engineering classifications, are presented on the Subsurface Data Sheet (Exhibit 4). SPT values used for calculation of internal friction angles were adjusted in a two-step process. First, the value was modified to account for the high efficiency (80%) hammer used in the test since N-ϕ correlations are based on older 60% efficiency hammer systems. Then the values were normalized to a constant effective stress of 1 atmosphere, creating a N1(60) equivalent. These results are shown in Appendix C.

5. SUBSURFACE CONDITIONS

Detailed information describing the soil conditions encountered during the subsurface investigation are presented on the Subsurface Data Sheet (Exhibit 4). In general, the borings encountered similar soil conditions. Specifically, below either asphalt pavement or a few inches of black, silty-clay topsoil, there was 8 to 10 feet of fill consisting of silty clay, silty sand and gravel. Non-cohesive fill soils tended to be loose to medium dense and the fine-grained soils soft to medium stiff. Below the fill is soft to hard, brown and gray, lean clay that extends to about 35 feet bgs. The clay is usually underlain by medium-dense sand with gravel, followed by medium-dense to dense, poorly graded, sand and gravel, which extend to the termination depth. The top of sand elevation was plotted at the current and 1962 boring locations. Contouring was used to estimate the clay/sand interface surface configuration, as shown in Table 6.

Table 6: Clay/Sand Interface

Substructure	Top of Sand Surface Elevation
Abutment 1	407.5
Pier 1	409.0
Pier 2	410.5
Pier 3	410.0
Pier 4	409.0

The borings produced samples at discrete locations within the subsurface environment beneath the site, and test data, whether insitu or exsitu, are representative only of those locations. To evaluate the general characteristics of the subsurface units, the data collected during field and laboratory testing were examined by observation, supported by statistical methods.

5.1 Man-made Fill

Fill consists of a variety of cohesive and non-cohesive soils. The unconfined compressive strength of near surface clay fill tends to be relatively high, as indicated by penetrometer testing in the field, suggesting that the material has been subjected to some compactive effort. The properties of the fill will have only minor impact on the design of pier foundations, which are usually placed 5 feet or so below existing grade. Fill properties below about 5 feet are characterized as loose, silty sand for purpose of pier and shaft design, with an estimated effective internal friction angle of 31 degrees (°), based on the SPT derived values presented in Appendix C.

The abutment design is more sensitive to the presence of fill because of the potential for slope instability, and for settlement with associated downdrag on piling. For assessment of these effects and the design of abutment foundations, the fill is assumed to be soft to medium-stiff, silty clay (Boring 4B-248).

5.2 Flood Plain Deposits (Clay)

This stratum is a significant member of the overall soil profile, capping the deeper outwash sands with about 25 feet of near-normally consolidated clay. The material tends to be silty clay, and unconfined compression testing suggests a highly variable strength profile. However, it is possible that this variability is more a reflection of the silt content than the actual strength of the material; silty samples with low cohesion tend to yield low apparent unconfined, compressive strength.

An analysis of the unconfined, compressive strength test results shows no significant correlation of strength with depth but the mean shear strength is similar from boring to boring. The mean shear strength (based on 50% of the unconfined, compressive strength) is calculated to be 871 psf, however the standard deviation is high (534 psf). Using a ?t' test produces 90% confidence that the mean is greater than 377 psf. By eliminating two high outliers the standard deviation is reduced and the 90% confidence limit mean increases to 490 psf.

Theoretically, the shear strength of normally consolidated clay should increase in a generally linear fashion with depth, proportional to the effective stress under which it has consolidated. Using this approach, a shear strength profile of 262 psf at a depth of 10 feet (base of the fill) increasing to 814 psf at the base of the clay layer would be predicted. The mean shear strength would be 538 psf, generally consistent with the means of the measured values.

As indicated above, the silt content of the soil is likely to play a large role in determining the unconfined compressive strength. This effect is minimized in the consolidated-undrained triaxial test and in fact, the siltier the sample the higher ϕ' is likely to be. Results of such a test conducted on a sample from Boring 4B-248 and tests preformed on generally similar material from other borings yielded the following results:

Table 7: CU Triaxial Test Results

Boring	Classification	Effective Friction Angle	Effective Cohesion
1W-66	A-6(8)	$\phi'=31.2°$	c'=190 psf
2B-106	A-6(15)	$\phi'=31.4°$	c'=240 psf
4B-248	A-6(9)	$\phi'=32.1°$	c'=0 psf

5.3 Sands

Sands extend beneath the floodplain clays to the total depth investigated. They comprise a variety of gradations and silt content with silty sands at one end of the spectrum and gravel (less frequently) at the other.

Effective stress parameters were estimated using corrected SPT blow counts – N1(60) values as presented in Appendix C. The values of ϕ' (effective internal friction angle) ranges from a high of 49° to a low of 30° throughout the area investigated. Since these values are all derived from N values, the variability must be examined in that context. For example, the highest ϕ' value is in a gravel (GP-GM) sample as may be expected (ϕ' = 49°). More surprisingly, the second to highest is a silty sand (SM) sample ϕ' = 45°. This result suggests that the presence of occasional large gravel particles could be having a significant impact on the raw N values, which is then reflected, in the calculated friction angles. Conversely, variations in the water level in the borings change effective stress conditions and may lead to under reporting of N, as appears to happen in the deeper part of Boring 4B-250.

An analysis of the friction angle data set for the four borings showed no significant correlation of ϕ' with depth and indicated that the results as a whole were near-normally distributed. The overall mean value of ϕ' is 34.6°, although this is higher than the individual mean in Boring 4W-270, suggesting that it would be a non-conservative value to use in design. The 90% confidence level mean is 33.7°, the value recommended for use in design.

The base of the outwash deposits was not determined with certainty, but nearby Boring B-247 was terminated at refusal at a depth of 93.6 feet (elevation 345.46 feet), which may be the top of bedrock. Other borings in the area (e.g. Boring 4B-250) were extended to elevations as low as 327.57 feet without encountering refusal. It should be noted that regional geological analysis suggests a valley floor at about elevation 335 feet. However, elevations could be substantially higher depending on proximity to the valley wall, or deeper where there are local irregularities.

5.4 Rock

Rock compressive strength testing (q_u) was conducted on samples from a nearby boring 4B-252 to support possible design of drilled shafts founded on rock. The rock appears to be dolomite based on its position in the geological column, and samples were tested at elevations 348.5 feet, 344.0 feet and 336.5 feet. The results varied from 8660 psi to 14730 psi with a mean of 11067 psi. Using a ?t' test, the mean is estimated to be greater than 5632 psi at a 90% confidence level.

5.5 Groundwater

FMSM installed groundwater observation wells in Borings 3B-177 and 3B-190 drilled for other bridges, and provided groundwater fluctuations during an established period. Groundwater levels fluctuated between elevations 420 to 423 feet. Groundwater was encountered in the borings at depths between 27 and 40 feet, corresponding to elevations of 413 and 405 feet. These elevations correspond roughly with the base of the lean, clay layer, which serves as a confining layer. Stabilized readings were not taken after completion of drilling. It should be noted that groundwater levels fluctuate significantly throughout the year with springtime increases that may be measured in tens of feet. The Federal Emergency Management Agency (FEMA) flood map for Louisville shows a 100-year flood elevation of 450 feet msl, which would inundate the bridge site to a depth of about 6 feet.

5.6 Selection of Design Parameters

Examination of the soil conditions along the bridge alignment suggests that the westerly end of the project (Boring 4W-270) is underlain by slightly better conditions (less fill and floodplain deposits) than are present elsewhere. Accordingly, for purposes of design, the substructures have been divided into two groups: Abutment 1 (A1) and Piers 1-4 (P1P4).

The soil strength profiles for these two design cases have been generalized in Table 8. Depths are below ground surfaces.

Table 8: Soil Strength Profiles

Profile	Depth (feet)	Soil	
Abutment 1 (A1)	0-7	silty clay (fill)	$\bar{S}_u$=500 psf
	7-29.5	silty clay	$\bar{S}_u$=500 psf
	29.5-116	fine-medium sand	$\bar{\phi}'$=33.7°
Piers 1-4 (P1P4)	0-10	silty sand (fill)	$\bar{\phi}'$=31.0°
	10-36	silty clay	$\bar{S}_u$=500 psf
	36-116	fine-medium sand	$\bar{\phi}'$=33.7°

Deep foundations may encounter bedrock at depths in excess of about 80 feet.

Design of foundations should consider that groundwater levels are likely to be at, or near, normal (elevation ~ 421 feet) during construction, but that seasonal increases will occur such that groundwater levels will be at the ground surface for extended periods. This provides the worst-case scenario for axial resistance analysis of deep foundations.

6. SETTLEMENT AND GLOBAL STABILITY EVALUATIONS

The west end of the planned bridge is supported on an abutment behind which there will be approximately 14 feet of new embankment. Settlement of the foundation soils under embankment loads may create differential movement at the transition from bridge to embankment, and may also create downdrag forces on deep foundation elements. Analyses were conducted to estimate the magnitude of settlement-related design constraints at the abutment.

The upper 29 feet of foundation soil at the abutment is fine grained and much of it (the lower 22 feet) is normally consolidated. The upper 7 feet is man-made fill and the stress history is not known. It is likely that some compaction occurred during placement and that subsequent land use imposed stresses in excess of those that would be expected in normally consolidated material. The deeper soils are sands and are subject to elastic compression under increased load.

Settlement estimates for the fine-grained soils were made using the computer program EMBANK (FHWA 1992) and the elastic settlement of non-cohesive soil was estimated primarily using the Hough method (LRFD BSD), discussed below. Results of these analyses are presented in Table 9.

Table 9: Abutment 1 Settlement Estimates

Stratum	Settlement (inches)	Notes
fill and floodplain silty clay	14.1	fill is normally consolidated
	9.0	fill over-consolidated 1,000 psf
outwash sands	0.6	Hough's method
	1.1	Schmertmann (1970)
	1.2	simple elastic calculation with Boussinesq stress distribution

Clearly, the fine-grained soils are the source of the major portion of the settlement, representing more than 90% of the movement that may be expected, regardless of the stress history of the existing fill. However, the estimated settlement of the sands could be critical in this design because of the potential for downdrag loads on deep foundations. Hence, the use of multiple techniques to assess whether the Hough method produces a conservative result (as it is often believed to) or not. Alternative methods suggest that the compression of the sand, while still small compared to the cohesive soils, could be greater than that predicted by Hough's method. According to LRFD BDS, the frictional resistance of materials in the zone above a point on the pile where settlement of the foundation soils is equal to 0.4 inches or greater must be ignored, and computed as a downdrag load. It is therefore important to locate the depth at which 0.4 inches of settlement occurs as accurately as is practicable. The maximum depths at which 0.4 inches of settlement is calculated to occur using the above three methods are as follows:

- Hough 37 feet
- Schmertmann 73 feet
- Boussinesq 70 feet

The significance of these values is that, first, there is a wide spread in the results and fine-tuning the analyses is not likely to reduce it much. Second, if the larger values (i.e., greater depths) are to be believed, then deep foundations that rely on skin friction for geotechnical resistance may not be feasible. Even the Hough-derived value of 37 feet will carry a substantial downdrag load/lost capacity penalty. This topic is pursued further in the section on deep foundation design.

The rate of settlement for the clay will be quite slow: 90% of the movement is expected to take several years. The sand will compress quickly with movement largely complete soon after construction of the embankment. This points to perhaps the simplest method for reducing downdrag-related loads and resistance penalties associated with the sand. If the embankment can be constructed before the deep foundations are installed, subsequent settlement associated with the sands can be virtually eliminated and with it the portion of the downdrag and capacity reduction associated with the sands. This may require temporarily building the embankment and then removing a portion of it in order to construct the abutment foundation, but there would be no need for an extended 'preload' period as there would if consolidation of the cohesive soils were the objective.

There is no simple way to mitigate the relatively large total settlement that is anticipated for the embankment and the associated differential movement between the fill and the abutment. Preloading is unlikely to be feasible because of the extended time that would be required. Lightweight fill may be too buoyant during anticipated floods. Wick drains may be a possible means for accelerating settlement but are still unlikely to produce the desired results within an acceptable time frame. Stone columns may be a feasible method of providing more stable embankment support. Alternatively, moving Abutment 1 to the west about 90 feet down the ramp to station 423+00 (i.e., add a span to the structure) would eliminate the need for new fill and solve both the downdrag and settlement problems.

The 12-inch, combined sewer located behind Abutment 1 and under the fill will likely suffer significant vertical movement, the actual magnitude of which will depend on its depth. Relocation of the sewer may be the best option, although moving the abutment, as suggested above, would eliminate the need for mitigating the vertical movement of the sewer.

The embankment height is less than 20 feet and does not trigger the requirement for global stability analysis (KYTC Geotechnical Manual GT-601-5); it is considered stable.

7. FOUNDATION ANALYSES AND RECOMMENDATIONS

7.1 General

Two alternate four-span structures are under consideration for the final design of Bridge B3RD-1, but foundation selection is unlikely to be affected by this choice. One end of the structure connects to an elevated span of Bridge 3RD-8, so only one abutment is required near the ground level terminus of Ramp 42. The bridge plan for the recommended structure is presented in Exhibit 2.

The presence of a substantial thickness of compressible clay at the site effectively eliminates consideration of shallow foundations for the abutment or the piers because of concern for excessive settlement. Of the deep foundation alternatives, driven steel friction H-piles would normally be the most likely choice. However, the need for the abutment foundations to resist quite severe downdrag loads requires that the possibility of using end bearing H-piles be considered too, as discussed below.

In this section of the report the axial geotechnical resistance of friction H-piles is addressed first. This sets the stage for discussion of pile load conditions, especially the impact of downdrag loads on the abutment foundations caused by the construction of the approach ramp embankment. Possible means for mitigation of these conditions are then presented, and the process is repeated for a drilled shaft alternative design. Three sizes of friction H-piles (12x53, 14x73, and 14x89) were analyzed for static and dynamic axial-bearing capacity, for pullout axial capacity and for pile drivability. Geotechnical resistances of four diameters of drilled shafts (30”, 36”, 42” and 48”) were estimated.

The LRFD design method requires independent consideration of the reliability of loads and resistances, and the assessment of structure performance under various limit states (e.g., service, strength). For geotechnical resistance [or ?load bearing capacity’ as it is known under the Allowable Stress Design (ASD) system], this is accomplished through the use of resistance factors that are applied to the nominal (or ultimate) resistance estimates to yield ?factored resistance’. These should not be confused with ?allowable load-bearing capacities’. They must only be used by the Designer in the context of an LRFD design and with appropriately factored loads.

7.2 Pile Axial Capacity

7.2.1 Friction H-Piles

Driven pile axial capacity evaluations were performed using the Tomlinson method for clays and Nordlund/Thurman method for sands to derive the nominal side resistance and nominal end bearing. Both methods are incorporated in the computer software APILE plus 4.0 (ENSOFT Inc., 2004). The total, nominal, geotechnical axial resistance at a particular depth is the sum of the nominal side resistance and nominal end bearing at that depth. The total, factored, geotechnical axial compressive resistance for static analysis was derived by multiplying the corresponding total nominal resistance by a resistance factor of 0.35 for clays and 0.45 for sands. The total, factored, geotechnical axial compressive resistance for dynamic analysis was derived by multiplying the corresponding nominal resistance by a unique resistance factor of 0.65 for both clays and sands. The total, factored, geotechnical axial uplift resistance was obtained from the nominal side resistance multiplied by a resistance factor of 0.25 for clays and 0.35 for sands. These factors are specified in LRFD BDS Table 10.5.5.2.3-1.

Axial capacity evaluations for three selected sizes of steel H-piles (12x53, 14x73, and 14x89) were performed for the project based on the assessment of soil conditions along the bridge alignment. The results of pile evaluations for each foot of depth below the bottom of the pile cap are presented in the tables included in Appendix D. It is to be noted that axial compressive resistance values in Appendix D are based on the assumption that positive skin friction develops along the entire pile length. Pile capacities have been calculated for Piers 1-4 as a group based on generalized soil conditions that were described in Section 5.

According to KYTC policy (August 18, 2006), maximum factored geotechnical resistance is limited as follows for 50 kips per square inch (ksi) steel H-piles:

HP 12x53 - 200 kips (100 tons)
HP 14x73 - 280 kips (140 tons)
HP 14x89 - 340 kips (170 tons)

Table 10 summarizes the required pile lengths and pile tip elevations as estimated for the maximum-factored, geotechnical resistance indicated above for each of the three pile sizes. Should more (or less) capacity be required, refer to the pile capacity tables in Appendix D. H-pile lengths at each substructure location should be adjusted accordingly, based on the actual pile driving conditions encountered in the field and the designed pile cap elevations.

It should be noted that nearby drilling encountered refusal at an elevation above some of the predicted pile tip elevations presented in Table 10 (elevation 345.46 feet); regional analysis of the bedrock conditions suggest that the valley floor may be encountered at about elevation 335 feet. Therefore the possibility of meeting refusal while driving piles to the stated depths cannot be ruled out. Where this occurs, it may be assumed that the design geotechnical axial compressive resistance of the pile has been attained.

Table 10: Estimated H-Pile Lengths for Maximum Factored Geotechnical Resistance, Based on Static Analysis Method.

Substructure	Pile Size	Maximum Total Factored Geotechnical Axial Compressive Resistance		Estimated Pile Length(1)	Estimated Pile Tip Elevation	Total Factored Geotechnical Axial Uplift Resistance(2)	
		(kips)	(tons)	(feet)	(feet)	(kips)	(tons)
Abutment 1	HP 12x53	200	100				
	HP 14x73	280	140	see	discussion	below	
	HP 14x89	340	170				
Piers 1-4	HP 12x53	200	100	85	354	148	74
	HP 14x73	280	140	95	344	208	104
	HP 14x89	340	170	108	331	252	126

(1) Below the bottom of the pile cap.
(2) Estimated uplift resistance for the corresponding estimated pile length.

7.2.2 Abutment Foundations

Standard Driven H-Piles

Absent downdrag loading, a friction H-pile foundation design for the abutment would be generally similar to that shown above for Piers 1-4. The piles would be a little shorter because the soil profile is slightly more favorable, but for the purpose of this discussion it is assumed that they are the same.

As described in Section 6, the soils beneath the abutment are compressible and subject to an increase in effective stress caused by the placement of 14 feet of fill. The resulting movement is likely to result in additional loading to deep foundations and these downdrag loads must be accommodated in the design of substructure components.

LRFD BDS requires that the downdrag load be calculated as the frictional resistance existing above a point on the pile where the soil settlement is equal to or greater than 0.4 inches. Further, the geotechnical resistance of this part of the pile is to be ignored. The approach discussed in Section 6 indicates that the 0.4-inch settlement point is estimated to occur at a depth between 37 and 73 feet.

Analysis of the 85-foot long HP 12x53 pile described in Table 10 using the neutral plane method, suggests that the neutral plane is 50 feet below the ground surface – somewhere near the mid-point of depths predicted by the settlement analyses. The neutral plane is located at a depth where the downdrag load approaches zero, and geotechnical resistance can begin to take effect. The analysis assumes that the maximum factored resistance of 100 tons is imposed, 90% of which is dead load and 10% live. Using a load factor of 1.25, the actual dead load would be 72 tons (or 144 kips).

With a neutral plane at a depth of 50 feet, the nominal geotechnical resistance for the pile between 50 feet and 85 feet is 150 tons, and the downdrag is 90 tons. The factored geotechnical resistance (using 0.45 for sand) is 67.5 tons, considerably less than the unfactored downdrag. Analysis of the HP 14x89 pile produces similar results: a factored geotechnical resistance of 110 tons that is less than the unfactored downdrag of 143 tons.

It must therefore be concluded that, based on LRFD BDS requirements for accommodating downdrag, standard friction H-piles cannot be used to support the abutment. It should be noted that if the abutments are not supported on friction H-piles, their possible use in the pier foundations must be reviewed carefully to ensure compatibility with the system that is ultimately selected for the abutment.

Alternative Foundation Systems

Relocation As indicated in Section 6.0, the most effective mitigation from a geotechnical standpoint is to relocate the abutment about 90 feet to the west in order to eliminate the need for fill. This in turn would eliminate the downdrag forces thereby making feasible the use of standard friction H-piles of a similar size and length as those used for the piers.

Ground Improvement Another approach to reducing settlement is to improve the soil conditions beneath the fill. This could be accomplished by using stone columns, controlled modulus columns, or one of the other hybrid systems that essentially provides a dense grid of support throughout the foundation footprint. The columns work by effectively supporting the embankment on the underlying sands, thereby relieving the stress in the compressible clays. These techniques are increasing in popularity as their effectiveness is demonstrated on similar projects. Specialty contractors typically specify and build the systems based on their proprietary designs and equipment. Lower costs may be achieved if there are multiple applications for this technique throughout the interchange and economies of scale can be realized. Through use of this technology, the need for a deep driven foundation might even be eliminated.

Precompression In Section 6, the possibility of reducing downdrag by accelerating elastic compression of the non-cohesive soils was raised. This involves construction of the embankment in advance of deep foundation installation. In this scenario, the outwash sands become the equivalent of an incompressible layer, and the neutral plane can be assumed (for simplicity) to be at the base of the overlying compressible soil. As a result, two improvements in pile performance can be anticipated. First, the length of pile subject to downdrag and for which no geotechnical resistance can be claimed is reduced, and second the magnitude of the downdrag itself is reduced. The results are illustrated in the following table.

Table 11: Downdrag Effects – Precompression of Sands

Pile Size	Table 10 Maximum Factored Geotechnical Resistance (kips)	Length (Ft.)	Downdrag Load (factored) (kips)	New Factored Geotechnical Resistance (kips)	"Capacity" Reduction[1] (%)
HP 12x53	200	85	69	113	43.5
HP 14x73	280	95	81	182	35.0
HP 14x89	340	108	83	236	30.6

(1) Considers downdrag load and reduction in frictional resistance.

These values are not intended to be used in design, but are illustrative of the reduced impact of downdrag after precompressing the sands. The % reduction is the impact on overall load bearing capacity caused by downdrag effects. Note that the reduction without precompression is more than 100% as discussed above, so this represents a significant improvement. On this basis, it appears that driven friction H-piles are a feasible foundation type subject to appropriate construction sequencing and compensation for the reduced efficiency of each pile. If this approach is adopted, individual piles may be designed as described below.

Static Analysis Method

$$RC = (RF_z - RDD + RT_z)\,(0.45)$$

and

$$RUL = (RULF_z - RUL_{DD})$$

where RC is the maximum factored geotechnical compressive resistance for a pile of length z ft., (kips), RUL is the maximum factored geotechnical uplift resistance for the same pile, RF_z is the nominal side (friction) resistance at depth z from the appropriate Appendix D Table, RDD is the nominal downdrag from Table 12 below (kips), RT_z is the nominal tip resistance at depth z from Appendix D, $RULF_z$ is the factored uplift resistance at depth z feet from Appendix D, RUL_{DD} is the factored uplift reduction due to downdrag* from Table 12, and 0.45 is the axial compressive resistance factor for sand. [*Note that this reduction is added back, in part at least, as a factored load in the downward direction during calculation of uplift resistance.]

Table 12: Downdrag Parameters

Pile Size	Nominal Downdrag RDD (kips)	Factored Uplift Reduction RUL_{DD} (kips)
HP 12x53	49	13
HP 14x73	58	15
HP 14x89	59	17

Dynamic Analysis Method

Compressive resistance computed using the dynamic analysis method produces a higher level of confidence in the predicted capacity, but must still be discounted for the effect of downdrag. Values presented in Appendix D may be adjusted as follows:

$$RC_{DYN} = (R_{DYNz} - RDD_{DYN})$$

where R_{DYN} is the maximum factored axial geotechnical resistance (dynamic analysis method) for a pile of length z ft., R_{DYNz} is total factored geotechnical axial compressive resistance (dynamic analysis method) at depth z from appropriate Appendix D table, and RDD_{DYN} is the factored downdrag from Table 13 below.

Table 13: Dynamic Method Downdrag Parameters

Pile Size	Factored Downdrag RDD_{DYN} (kips)	Unfactored Downdrag Load (kips)
HP 12x53	37	57
HP 14x73	44	68
HP 14x89	46	71

The unfactored downdrag loads should be appropriately factored and added to other loads applied to the pile.

Bituminous Coatings In the class of solution that deals with the drag load, there are two approaches to minimizing its impact. First, this load can be significantly reduced by coating the pile with bitumen in the downdrag zone. This has been known to reduce downdrag by anywhere from 50% to 90% in tests. Bitumen creates a low-shear strength zone that is estimated to be on the order of 200 psf (FHWA, 1998). The clays primarily responsible for creating the shallow downdrag forces have an average shear strength of 500 psf. Downdrag would therefore be effectively reduced by about 60%. The reduction in the sands theoretically could be greater, but it may be difficult to guarantee the integrity of the bitumen coating during driving through sand; it would be prudent to just accept the more conservative reduction value in the clay. However, there is still a large loss of geotechnical resistance because only that below the 0.4-inch soil settlement mark (or the neutral plane) may be considered. The net result is estimated to be, for example, a 108-foot long HP-14x89 with a factored geotechnical resistance of 110 tons and an unfactored downdrag of 57 tons. With a load factor of 1.25 this becomes an offset of 71 tons leaving fewer than 40 tons of 'useful' capacity in a pile that, as shown in Table 10, started life with 170 tons of factored axial resistance. If precompression of the sands is accomplished, as described above, 60% reduction of the remaining downdrag forces would substantially eliminate this capacity reduction.

Founding on Bedrock The second approach to downdrag mitigation involves extending the piles to bedrock. If as suspected, the bedrock horizon is at about 335 feet, it may be possible to drive the piles to bedrock and eliminate the reduction of capacity caused by the '0.4-inch settlement' rule. In other words, the example above would still retain a 170-ton factored resistance because support is not derived from frictional resistance above or below the 0.4-inch settlement mark. The downdrag magnitude would not be affected; if the pile were untreated, the downdrag would be on the order of 50 tons (unfactored). If bitumen coating were to be used, the downdrag would likely be on the order of 20 tons (unfactored) leaving more than 140 tons of useful resistance. If the abutment piles are to be driven to bedrock, consideration also should be given to driving all the piles to bedrock (piers as well) where differential vertical movement might be unacceptable.

7.2.3 Drilled Shafts

As an alternate to H-pile foundation support, a drilled shaft option was analyzed. Axial capacity evaluations were performed using the alpha-method for clays and beta-method for sands to derive the nominal side resistance, and the method developed by O'Neill and Reese (FHWA, 1999) was used to derive the nominal end bearing capacity. These analyses result in a shaft that derives the bulk of its support from frictional resistance in the soil column.

The following factors were applied to the nominal resistance in order to obtain the factored resistance: 0.45 for side resistance in clays, 0.40 for end bearing in clays, 0.55 for side resistance in sands, and 0.50 for end bearing in sands. The total factored geotechnical axial compressive resistance at a particular depth is the sum of the factored side resistance and factored end bearing at the corresponding depth. The total factored geotechnical axial uplift resistance was obtained from the nominal side resistance multiplied by a resistance factor of 0.35 for clays and 0.45 for sands (LRFD BDS Table 10.5.5.2.4-1). Where the upper 5 feet of a shaft was surrounded by cohesive soils, no frictional resistance was assumed to exist in this zone (FHWA, 1999).

Axial capacity evaluations for four drilled shaft diameters (30", 36", 42", and 48") were conducted for the Pier 1-4 soil profile. The results are presented in the tables included in Appendix E. It is to be noted that axial compressive resistance values in Appendix E are based on the assumption that positive skin friction develops along the entire shaft length unless the upper 5 feet of the shaft is in cohesive soil.

Design of a shaft-supported foundation for Abutment 1 involves consideration of embankment-induced downdrag, as was the case for the driven pile designs discussed above. For the purpose of this assessment, it is assumed that the geotechnical characteristics of the subsurface profile at the abutment are sufficiently similar to those at the piers for the same shaft design to be applicable (i.e. as presented in Appendix E).

A 30-inch diameter shaft is evaluated to determine the likely impact of downdrag. A 114-foot deep shaft (assumed to be founded in soil, not on bedrock) has a factored, geotechnical resistance of a little over 1200 kips if there is no downdrag effect. This means that the maximum factored load cannot exceed 1200 kips of which 90% is assumed to be dead load. If the load factor is 1.25, then the actual dead load is 864 kips.

Constructing a neutral plane diagram shows the plane to be 65 feet below ground surface. The associated downdrag is 320 kips (unfactored) and the nominal geotechnical resistance has now been reduced to 700 kips, or 316 kips factored. The shaft is therefore so inefficient that its use cannot be recommended for support of the structure. An analysis with similar results can be made for the other shaft diameters.

Solutions of the same general classes as for the driven pile system can be implemented, i.e., relocation of the abutment, ground improvement and end bearing on bedrock. Bituminous coating is not feasible, but other methods for creating a low-shear strength zone may be possible. Of these options, however, only end bearing on bedrock requires further elaboration.

The axial resistance of a shaft founded on (or socketed into) bedrock is a function of the compressive strength and structure of the rock formation providing support. In this case the rock type is not known with certainty. At nearby Boring 4B-252, was logged starting at an elevation of 354.7 feet msl (RQD 60 – 70%). Another boring in the area, (4B-247), encountered refusal at 345.5 feet msl. Near Pier 4, however, Boring 4B-250 was drilled to 327.6 feet msl without encountering refusal. Based on these elevations, the most likely rock type to be encountered would be dolomite.

Assuming a compressive strength of 5600 psi, the nominal geotechnical resistance of a 30-inch diameter shaft in a 45-inch deep rock socket will be about 3100 kips. With a resistance factor of 0.50, the factored resistance becomes 1550 kips. As indicated above, downdrag on this size of shaft is estimated to be on the order of 320 kips (unfactored), or 400 kips factored. This demonstrates the geotechnical feasibility of designing a reasonably efficient end-bearing shaft. The geotechnical resistance of shafts of various diameters is shown below, along with a general indication of the downdrag to be considered in design. Final values can only be provided once actual dead loads have been allocated to the shafts.

Table 14: Shaft-Resistance and Downdrag
(all in kips except shaft diameter)

Shaft Diameter (inches)	Nominal End Bearing Resistance	Unfactored Downdrag (no precompression)	Unfactored Downdrag (with precompression)	Axial Factored End Bearing Resistance
30	3088	282	61	1544
36	4445	339	74	2222
42	6050	395	86	3025
48	7903	452	98	3452

(Subject to confirmation that q

7.3 Pile Driveability

A driveability assessment was made to provide guidance on the size of hammer likely to be required for installation of H-piles to the specified resistances without over-stressing the pile on the one hand, or using excessive blow counts, on the other.

For purposes of the analysis, it was assumed that the Contractor will be installing piles with maximum-factored geotechnical resistance, as shown in Table 10. The soil profile representing Abutment 1 and Piers 1-4 was selected for analysis. The Delmag family of hammers was selected as representative of diesel hammer performance for the applicable energy levels.

Analyses were conducted using Wave Equation software (GRLWEAP) developed by Pile Dynamics, Inc. for 12x53, 14x73 and 14x89 steel H-Piles and using hammers with a range of driving energy from 20 ft-kips to 80 ft-kips.

The soils contributing to the driving resistance include the upper clay layer and the underlying sand layers. In order to account for soil remolding, pore water pressure increase and corresponding shear strength decrease, the following skin friction reduction percentages proposed by KYTC were used to calculate driving resistance: 50% for clay and 25% for sand. The driving resistances corresponding to the pile lengths in Table 10 can be found in Appendix D. For each pile section/hammer combination, the relationship between static soil resistance (kips) and dynamic driving resistance [blows per foot (bpf)] was developed and plotted graphically.

The hammer energy to adequately drive the piles without delivering excessive blow counts or overstressing the piles has been set on criteria established by FHWA (Soil and Foundation Workshop Manual) as blow counts between 30 and 144 blows per foot and maximum stress in steel less than 0.9 times the yield (45 ksi for Grade 50 steel). The results are shown in Exhibit 5. It appears that the 60 ft-kip and 80 ft-kip hammers are capable of driving all three sections to the maximum factored axial resistance without excessive blow counts and /or stress to the pile. The 60 ft-kip hammer appears suitable for 12x53 and 14x73 sections, but a larger hammer (at least 80 ft-kip) should be considered for 14x89 section H-piles.

The Designer may use the driving resistance table in Appendix D and Exhibit 5 to correlate minimum driving resistance required to achieve a desired factored geotechnical axial resistance for other pile lengths, as needed.

It should be noted that the information presented in Exhibit 5 is general in nature, and is not intended to replace independent analyses by pile driving contractors using actual hammer and driving accessory combinations. Further, the results of wave equation analyses should be calibrated based on dynamic field testing prior to production driving and reliance on the predictions of static axial load capacity.

7.4 Lateral Squeeze

Bridge abutments supported on piles driven through soft compressible cohesive soils may, under some circumstances, tilt because of lateral movement of the foundation soil associated with settlement strains. This phenomenon, known as lateral squeeze, is accentuated if there is a significant unbalanced load behind the abutment. The analysis of this potential condition is made using two rules of thumb for purposes of preliminary foundation evaluation.

First, lateral squeeze is not likely to occur if the vertical pressure exerted by the embankment behind the abutment is less than three times the undrained shear strength. At this structure, the embankment load is estimated to be 1820 psf. This suggests that if the foundation soil shear strength is less than 600 psf, lateral squeeze should be considered in design. In fact, the design shear strength for pile and shaft design is 500 psf. This is considered to be a conservatively low mean value but should, in any case, be cause for concern.

The magnitude of lateral squeeze displacement effects is assessed by the second rule of thumb, which suggests that horizontal movement may be as much as 0.25 times the vertical settlement. This could amount to 2 – 3 inches, with the top of the abutment tending to move away from the embankment.

There are various solutions to this problem. First, lateral squeeze is unlikely to occur once the settlement of the embankment is substantially complete, at which time the foundations could be installed. This may not be a remedy that can be employed here because of timing issues. Eliminating the problem through the use of lightweight fill, whereby the embankment load is reduced below 1500 psf, is likely to be feasible, provided the fill is still sufficiently dense to resist buoyancy forces. Relocation of the abutment, as discussed earlier, provides a positive and guaranteed solution.

Engineering solutions that cope with the problem include provision of large expansion shoes that can accommodate increased movement, and the use of steel H-piles (as opposed to concrete) that have high tensile strength in flexure. The latter point merely assists in preserving the load bearing ability of the foundation once movement has occurred.

7.5 Lateral Load Analysis Parameters

Deep foundation elements subjected to horizontal loads should be analyzed for maximum bending moments and lateral deflections. The required lateral load capacity can be obtained by increasing the diameter or the embedment depth of the foundation element. The Site Specific Idealized Soil Profile and corresponding recommended lateral soil modulus, and soil strain to be used to analyze the laterally loaded pile by the p-y curve method are shown in Appendix F. The p-y curve method for laterally loaded pile analyses is routinely performed using either the L-Pile Plus or Com 624 computer software.

7.6 Seismic Load Evaluation and Liquefaction Potential

The soil profile at the site corresponds most closely to Type 3 (LRFD BDS – Table 3.10.5.1-1) resulting in a Site Coefficient (S) of 1.5. (A profile with soft to medium stiff clays and sands characterized by 30 feet or more of soft to medium stiff clays with or without intervening layers of sand or other cohesionless soils.)

Liquefaction of saturated, loose cohesionless soil may occur in response to vibration such as occurs during earthquakes. The attempted movement of individual soil particles is constrained by interstitial water, the pressure of which rises for the duration of the shaking. This increase in pore pressure corresponds to a decrease in effective stress and a reduction of associated strength parameters critical to foundation stability. Because saturated, cohesionless soils are present beneath the B3RD-1 bridge site and USGS has established seismic design parameters for the area, a preliminary assessment of liquefaction potential was made.

The soils below a depth of about 35 feet bgs are predominantly silty sands, sands and gravelly sand mixtures. At this depth the soils are permanently saturated. The shallow surficial fill in the area is not dominantly cohesionless, but pockets of silty sand are present (e.g., Boring 4B-250). These soils are typically not saturated, but may become so during the annual increase in stage of the Ohio River. Accordingly, an assessment was made considering the soils at a depth of 6 feet with a groundwater table at the surface, and at 35 feet, which is below the permanent water table.

The analysis was conducted in accordance with the methodology proposed by Idriss and Boulanger (2004). In this method the cyclic shear ratio (CSR) is calculated and compared with the cyclic resistance ratio (CRR) computed based on corrected SPT 'N' values. The process is simplified by the use of graphical solutions to the underlying equations.

Results of the assessment are presented in Exhibit 6, and show that the potential for liquefaction of the deep soils is remote. While that of the shallow fill material is higher, it too is very low.

As indicated in Section 2.2, seismic design guidelines are currently under review by AASHTO; this section should be updated accordingly, if changes occur.

8. SUMMARY OF FOUNDATION RECOMMENDATIONS

8.1 General

1. The bridge foundation design is complicated by anticipated settlement of the proposed approach embankment that is expected to be on the order of 1 foot. This creates an interface problem where the embankment and abutment meet; differential settlement at this location should generally be kept to less than 1 inch. Settlement of the foundation soils also creates the potential for significant downdrag loads on deep foundation components.

   Mitigation of the differential settlement problem can be accomplished by relocation of the abutment about 90 feet west to eliminate the need for embankment fill. Alternatively, ground improvement in the form of stone columns or similar proprietary systems (e.g., constant modulus columns) could be undertaken in order to substantially reduce settlement and associated downdrag loads.

   Downdrag loads on both driven friction H-piles and drilled friction shafts are potentially severe and can render those foundation types infeasible unless special precautions are taken. These include:

   - Precompression of the sands through early construction of the embankment
   - The use of bitumen coating to reduce friction forces in the downdrag zone
   - Founding support members on bedrock

   These precautions each provide a benefit, as described in earlier sections, and can be employed either individually or collectively.

2. No embankments are planned in the vicinity of the piers and so settlement induced down drag is not a design consideration, however compatibility with the foundation at the abutment should be considered particularly if it is decided to found the support members on bedrock.

3. Based on the geotechnical information developed during this investigation, end-bearing H-piles appear to represent the best candidate for supporting the proposed bridge substructures. Shorter friction H-piles may be used for the piers, provided this is compatible with the abutment foundations.

4. Foundation analyses were performed according to the provisions of LRFD BDS.

8.2 Steel H-piles

1. Three sizes of Grade 50 steel H-friction piles (12x53, 14x73, 14x89) were analyzed for static and dynamic axial bearing capacity, for pullout axial capacity and for pile drivability.

2. Pile capacities were developed for two soil profiles, one representative of conditions at Abutment 1, and the other, at the piers. Axial capacity estimates for single, steel H-piles are provided in Appendix D. Upon determination of the final H-pile locations, arrangement and loads, the designer should use the capacity estimates to determine the H-pile size and length. The factored geotechnical axial resistance estimates provided in Appendix D are based on the LRFD BDS resistance factors.

3. If load testing and/or dynamic analysis of driven piles in soil is conducted, the LRFD resistance factors used to determine the factored axial capacity for design purposes may be revised as outlined in LRFD BDS Table 10.5.5.2.3-1, based on site variability and the number and type of tests performed. The Designer should note that lateral capacity requirements will need to be revisited if the shaft lengths are revised based on load testing and/or dynamic analysis.

4. All pile axial capacities presented in Appendix D are for single piles. In addition to applying appropriate resistance factors, individual capacities for piles in-group configurations may be further reduced depending upon soil type, bearing condition of the pile cap, or center-to-center spacing as recommended in LRFD BDS. The following criteria should be observed:

Table 15: Group Efficiency Factors

CTC Spacing	Group Efficiency Factor		
	Cohesive Soils		Cohesionless Soils (Cap in or not in firm Contact with Ground)
	Cap not in firm Contact with Ground	Cap in firm Contact with Ground	
6B	1.00	1.00	1.00
2.5B	0.65	1.00	1.00

The notation "B" is the shaft diameter and the percent reduction may be linearly interpolated between the values and spacing provided.

5. LRFD BDS include a resistance factor for horizontal geotechnical resistance of a single pile or pile group of 1.0 for lateral capacity analyses. Appendix F provides the recommended soil parameters for use in lateral load pile analyses.

6. For the initial group of piles installed, the piles should be left for a minimum of one day so excessive

pore pressure caused by driving operations can dissipate and the soils can set-up. After the one-day waiting period, the piles should be re-struck to see if the required bearing capacity was achieved. If the set-up effect is not significant, subsequent groups of piles will not require re-striking.

7. Hammer selection was based on the ultimate driving resistance that 12x53, 14x73 and 14x89 steel H-piles would experience during the installation process. The results of these calculations are presented in Exhibit 5.

8. Upon selecting the pile size and length required to support the applied loads, the Designer should select the appropriate hammer required to drive the piles to the specified depths from Exhibit 5. The Designer should place a note on the drawings that states: "A hammer system capable of delivering a minimum energy of ___ foot-kips will be necessary to drive the piles to the maximum total factored geotechnical axial resistance without encountering excessive blow counts and over-stressing the piles. The Contractor should submit appropriate pile driving systems to the Kentucky Transportation Cabinet for approval prior to the installation of the first pile. Approval of the pile driving system by the Engineer will be subject to satisfactory field performance of the pile driving procedures."

9. Drivability studies were performed assuming continuous driving. If interruptions in driving individual piles should occur, difficulties in continuing the installation process will likely occur due to the pile "set-up" characteristics.

10. LRFD BDS (Section 6.5.4.2) recommends the following resistance factors for determining the structural capacity of steel H-piles:

Table 16: Resistance Factors – H-Pile Structural Capacity

Loading Condition	**Resistance**	
	Piles Subjected to Damage From Severe Driving Conditions[1]	**Good Driving Conditions**
Axial Resistance In Compression	φ_c=0.50	φ_c=0.60
Combined Axial and Flexural Resistance	N/A	φ_c=0.70 φ_f=1.00

[1]Apply these values only to the section of the pile likely to be damaged during driving (LRFD BDS Section 6.15.2).

8.3 Drilled Shafts

1. This option has been evaluated for comparison with H-pile supported foundations.

2. Axial capacity estimates for drilled shafts are provided in Appendix E and were derived utilizing the LRFD resistance factors in LRFD BDS. The capacities presented are for single shafts and do not include group reduction factors. Upon determination of the final shaft locations and shaft loads, the Designer should use the capacity estimates to determine shaft size and length for each shaft.

3. If load testing of drilled shafts in soil is conducted, the LRFD resistance factors used to determine the factored axial capacity for design purposes may be revised as outlined in LRFD BDS Table 10.5.5.2.4-1.

4. All shaft capacities presented in Appendix E are for single shafts. In addition to applying appropriate resistance factors, individual capacities for shafts in group configurations should be further reduced depending upon center-to-center spacing as specified in the LRFD BDS. The following criteria should be observed:

Table 17: Drilled Shaft Group Efficiency Factors

CTC Spacing	Group Efficiency Factor for Cohesive Soils	Group Efficiency Factor for Non-Cohesive Soils
6B	1.00	N/A
4B	0.80	1.00
2.5B	0.65	0.65

The notation “B” is the shaft diameter and the percent reduction may be linearly interpolated between the values and spacing provided.

5. LRFD BDS calls for a resistance factor for horizontal geotechnical resistance of a single shaft or shaft group of 1.0 for lateral capacity analyses. Appendix F provides the recommended soil parameters for use in lateral load analyses.

6. The Contractor should embed the drilled shafts to the plan tip elevation or to an elevation as directed by the Engineer.

7. If a temporary casing for drilled shafts is used during construction, the Contractor should either wait until concrete has been placed for the entire length of the shaft before pulling the casing, or the level of the concrete being placed should be maintained several feet above the hydrostatic head as the casing is retrieved. These measures should be implemented by the Contractor to reduce the likelihood of soils collapsing into the shaft excavation and detrimentally affecting the structural integrity of the drilled shafts.

8. It is recommended that Class A Modified concrete, which is in accordance with the current KYTC Special Note for Drilled Shafts, be used in construction of the drilled shaft. The concrete should also exhibit good workability, i.e., high slump. Once an excavation is complete and the steel reinforcing cage has been placed, concrete should be tremmied to the bottom of the shaft and should replace/displace any water or slurry remaining after drilling operations.

9. If drilling slurry is to be used during shaft installations, the slurry should be capable of suspending the soil particles encountered and not leave a thick coating of slurry, or "mud", on the excavation sides or bottom. In accordance with the current "Special Note for Drilled Shafts", the Contractor shall submit a detailed plan for its use and disposal along with a drilled shaft installation plan to the Geotechnical Branch of the KYTC for approval prior to implementation. The Contractor shall supply all equipment and construction techniques involving slurry that are necessary to maintain environmental standards.

10. Drilled Shaft Integrity Testing will be required for each drilled shaft. An appropriate number of Crosshole Sonic Logging (CSL) access tubes (approximately 3), consisting of 2-inch nominal diameter schedule 40 steel pipes, will be required. These tubes should be shown on the drilled shaft details with the following note on the Drilled Shaft Detail Sheet:

> *Perform non-destructive Drilled Shaft Integrity Testing on the Drilled Shafts using Crosshole Sonic Logging (CSL) in accordance with the "Special Note for Non-Destructive Testing of Drilled Shafts". The Department will pay for this testing and associated cost at the contract unit bid price for "CSL Testing". This includes CSL Testing Mobilization and CSL Testing. The access tubes are incidental to the shaft.*

11. Unless otherwise specified, all construction methods and materials used for drilled shaft installations shall be in accordance with the current "Special Note for Drilled Shafts".

9. QUALIFICATIONS

This investigation was performed in accordance with accepted geotechnical engineering practice for the purpose of determining bridge foundation recommendations only. Verification of the subsurface conditions for purposes of determining contamination, difficulty of excavation, the effect of excavation on slope stability or existing structures, and trafficability is beyond the scope of this study. The analyses and recommendations submitted in this report are based upon the data obtained from the borings drilled at the locations shown on Exhibit 2, and on the Subsurface Data Sheet (Exhibit 4). This report does not reflect any variations that may occur between the borings or elsewhere on the site, and variations whose nature and extent may not become evident until a later stage of construction. In the event that any changes in the nature, design or location of the proposed bridges are made, the conclusions and recommendations contained in this report should not be considered valid until the changes are reviewed, and the conclusions and recommendations in this report have been modified or verified in writing by a geotechnical engineer.

It has been a pleasure to be of service to Kentucky Transportation Associates in performing this geotechnical investigation for Structure 0180, Bridge B3RD-1.

Respectfully Submitted,

BARR & PREVOST

Gary R. Simmons, P.E.	Stuart Edwards, P.E.
Principal	Senior Project Manager

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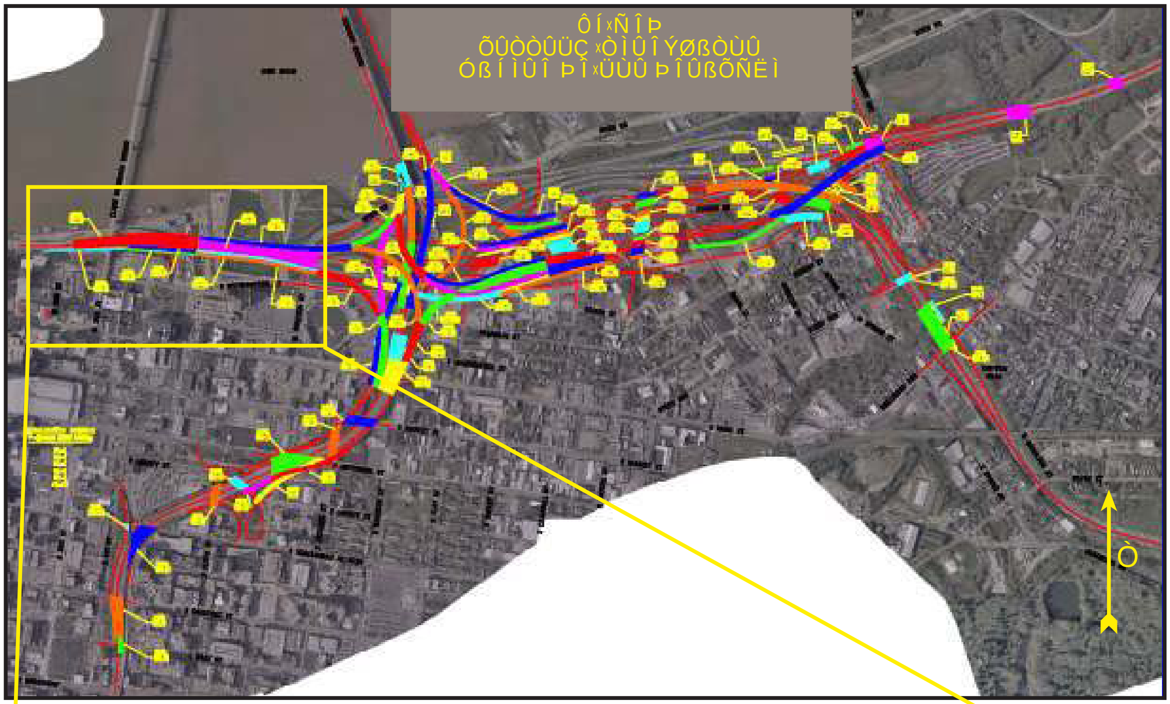
EXHIBITS 1-6



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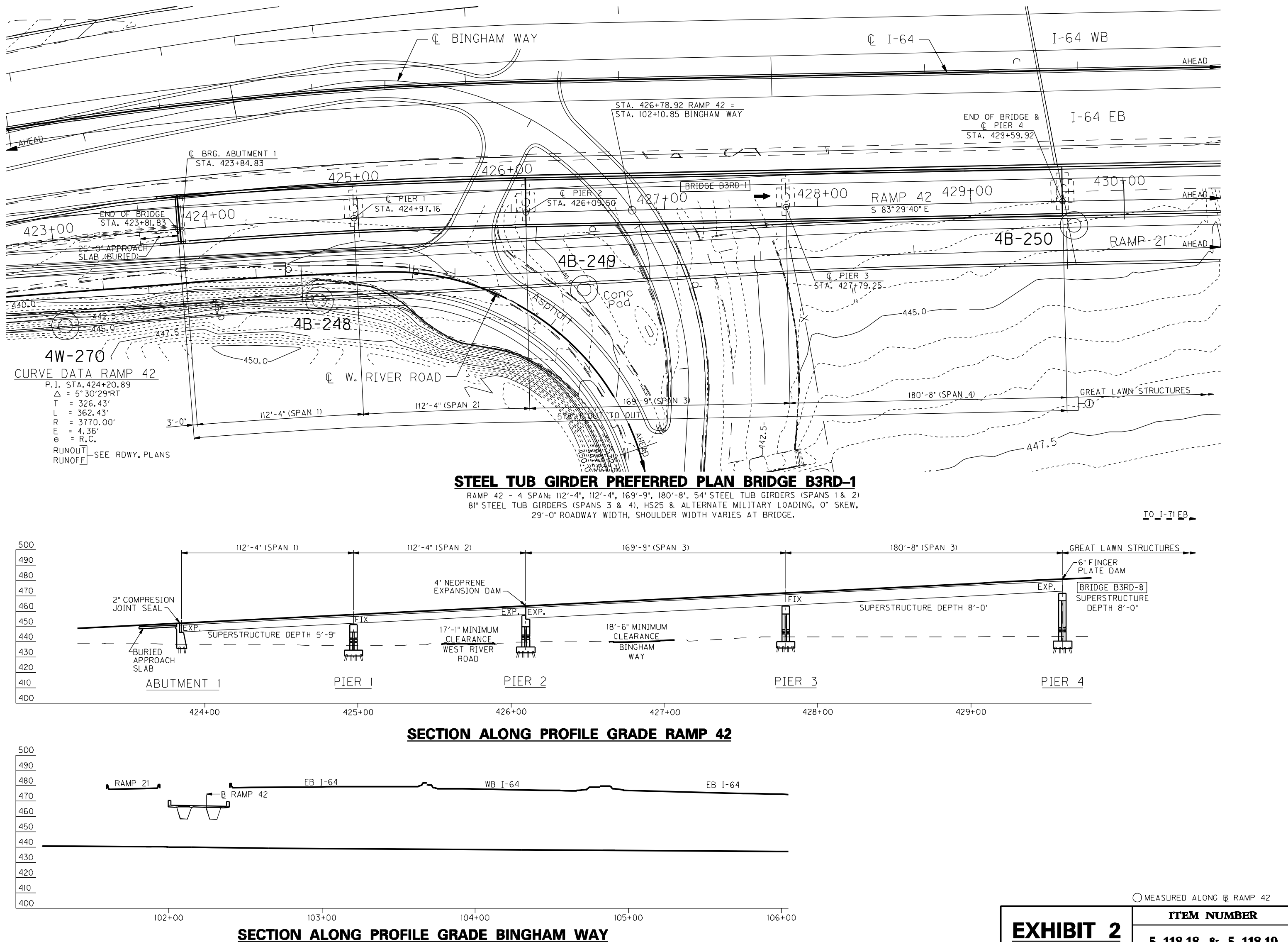


STEEL TUB GIRDER PREFERRED PLAN BRIDGE B3RD-1

RAMP 42 - 4 SPAN: 112'-4", 112'-4", 169'-9", 180'-8". 54" STEEL TUB GIRDERS (SPANS 1 & 2)
81" STEEL TUB GIRDERS (SPANS 3 & 4), HS25 & ALTERNATE MILITARY LOADING, 0° SKEW,
29'-0" ROADWAY WIDTH, SHOULDER WIDTH VARIES AT BRIDGE.

SECTION ALONG PROFILE GRADE RAMP 42

SECTION ALONG PROFILE GRADE BINGHAM WAY

LEGEND

- DENOTES EXISTING GROUND LINE
- BORING LOCATION

○ MEASURED ALONG ℄ RAMP 42

REVISION	DATE
DATE:	CHECKED BY
DESIGNED BY:	
DETAILED BY:	

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE I-64 | CROSSING WEST RIVER ROAD

PREFERRED LAYOUT BRIDGE B3RD-1

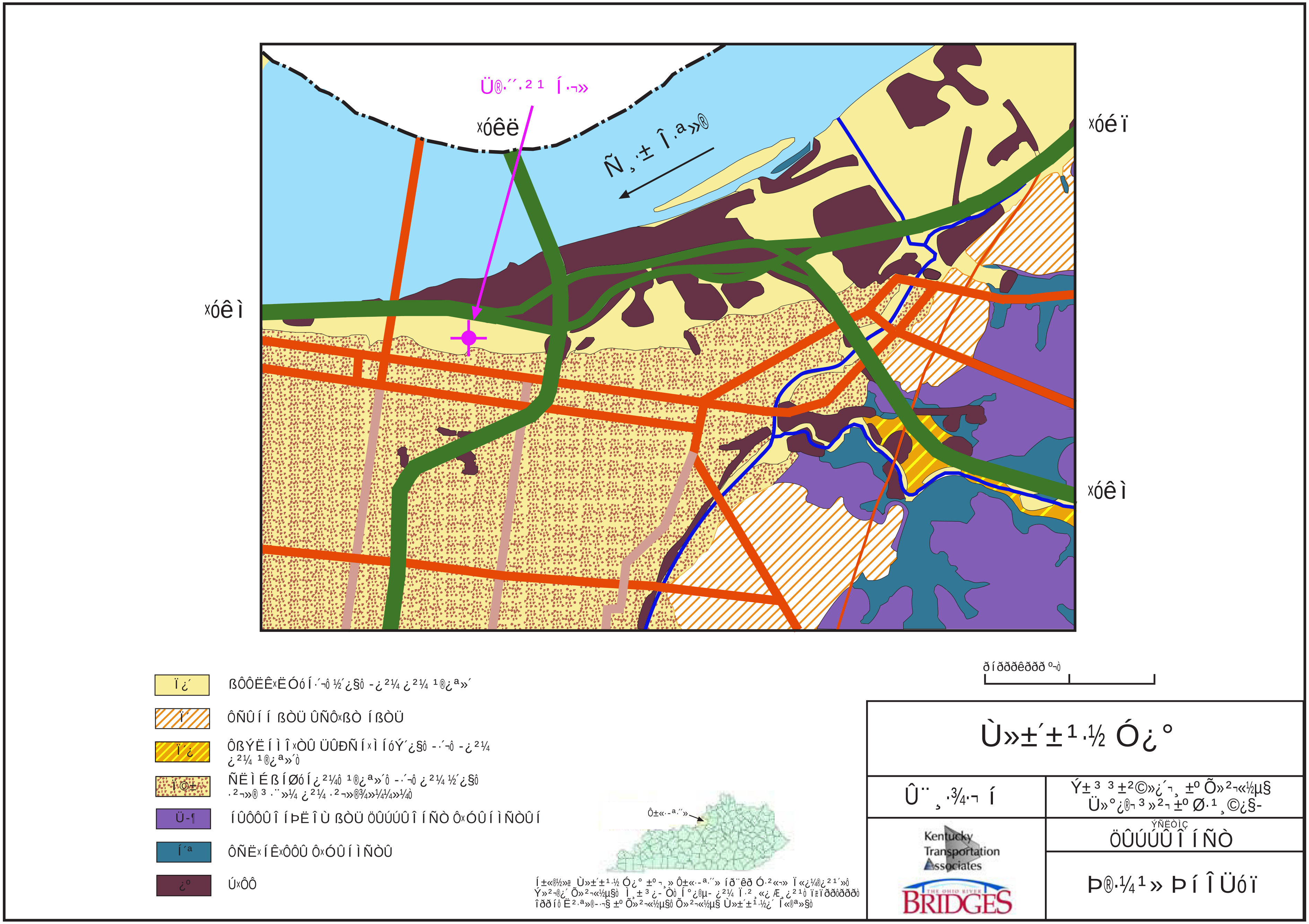
PREPARED BY
THE OHIO RIVER BRIDGES
Kentucky Transportation Associates

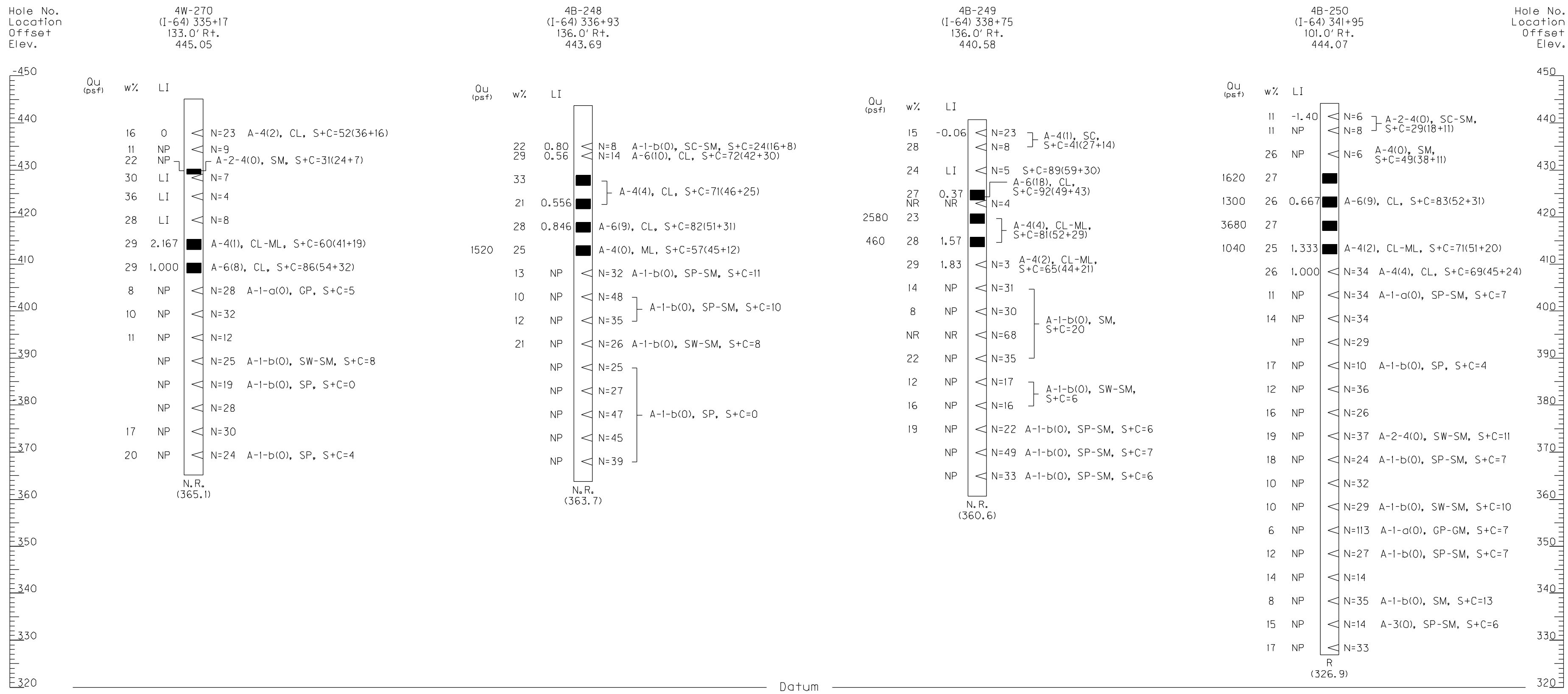
SHEET NO.
DRAWING NO.

EXHIBIT 2

ITEM NUMBER
5-118.18 & 5-118.19

SHEET LOCATION: FILE NAME: $$$$design$file$specification$$$$ USERNAME: $$$$USER$$$$ DATE: $$$$DATE$$$$ E-SHEET NAME:


Kentucky Transportation Associates
THE OHIO RIVER BRIDGES



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REVISION	DATE
DATE:	CHECKED BY
DESIGNED BY:	
DETAILED BY:	

Commonwealth of Kentucky
DEPARTMENT OF HIGHWAYS

COUNTY
JEFFERSON

ROUTE I-64 | CROSSING RIVER ROAD

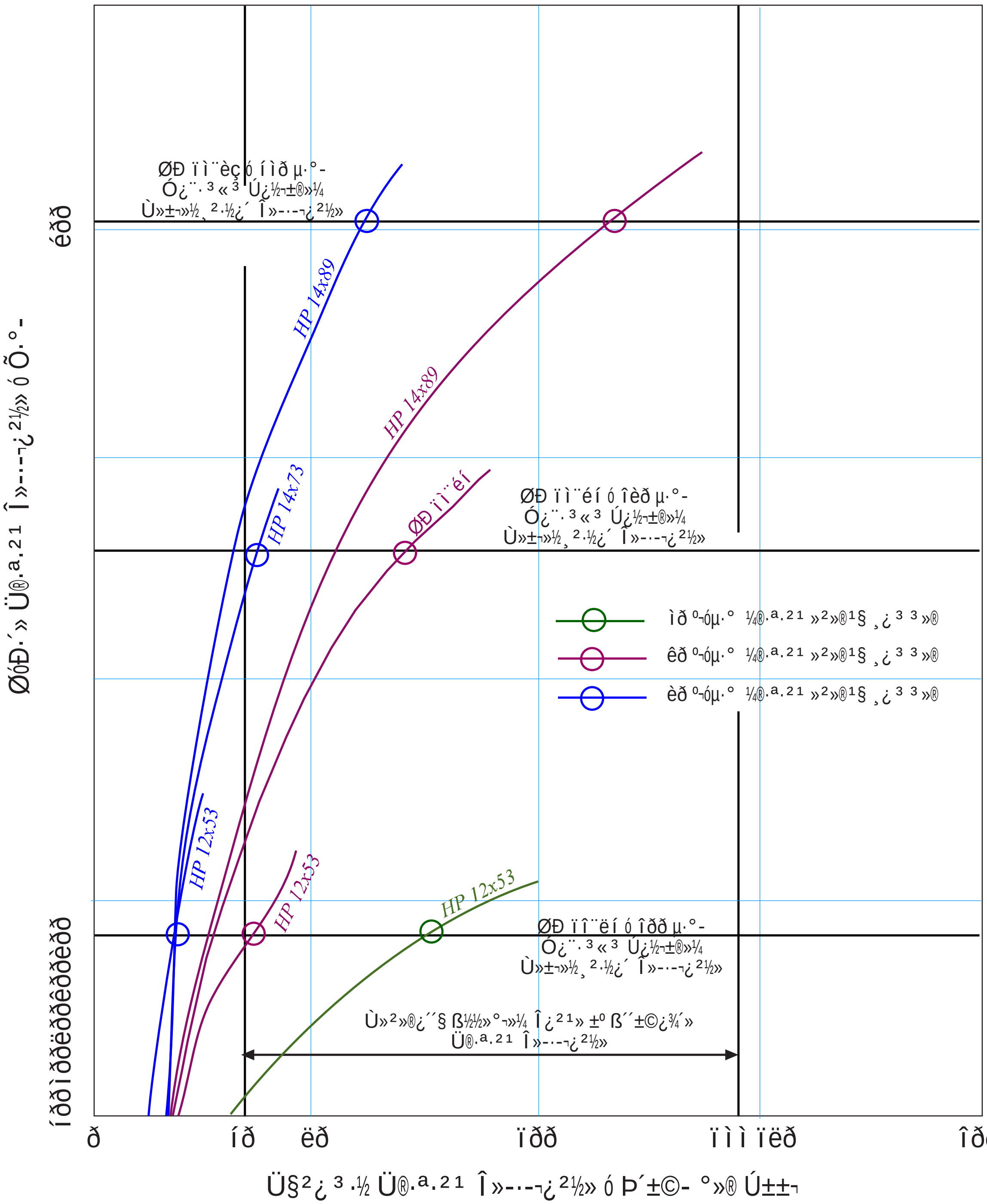
PREFERRED LAYOUT BRIDGE B3RD-1

EXHIBIT 4

ITEM NUMBER
5-118.18 & 5-118.19

PREPARED BY
Division of Structural Design
GEOTECHNICAL BRANCH

SHEET NO.
DRAWING NO.



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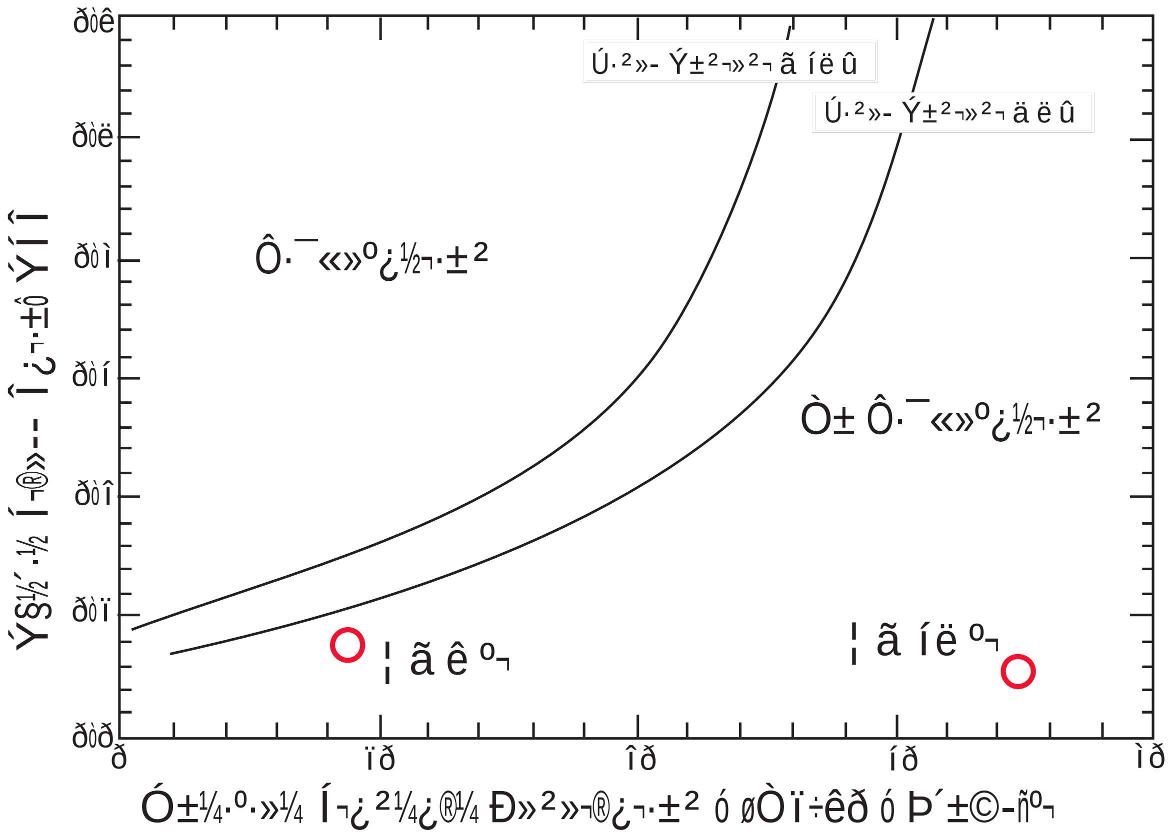

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APPENDICES
A - F

APPENDIX A
COORDINATE DATA SUBMISSION FORM

COORDINATE DATA SUBMISSION FORM
KYTC DIVISION OF STRUCTURAL DESIGN -- GEOTECHNICAL BRANCH

County Jefferson
Road Number I-65
Survey Crew / Consultant QK4 / Barr & Prevost Inc.
Contact Person Gary Simmons
Item # 5-118.18 & 19
MARS # C-04224166
Project # S0180 Bridge3RD-1

Date 1/9/2006

Notes:
All coordinates should be NAD-83 Latitude, Longitude in Decimal Degrees

(Please Mark One)
Elevation Datum **Sea Level** **Assumed**

BORING NUMBER	STATION	OFFSET	ELEVATION (ft)	LATITUDE	LONGITUDE
4B-248	336+93	136' Rt	443.69	38.2586517780	85.7510180750
4B-249	338+75	136' Rt	440.58	38.2586164020	85.7504175910
4B-250	341+95	101' Rt	444.07	38.2586266480	85.7492946120
4W-270	335+17	133'Rt	445.05	38.2586600810	85.7515985290
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APPENDIX B
CONSOLIDATION AND STRENGTH TESTING LABORATORY RESULTS

Wang Engineering, INC.
Geo-Environmental Engineers
1145 N Main Street
Lombard, IL 60148
Phone : 630 953-9928
Fax: 630 953-9938

Project Name: Kennedy Interchange - Phase 4 - S0160
Source: 4B-248 #4, 20'-22'
Cv computation Method: Cassagrande

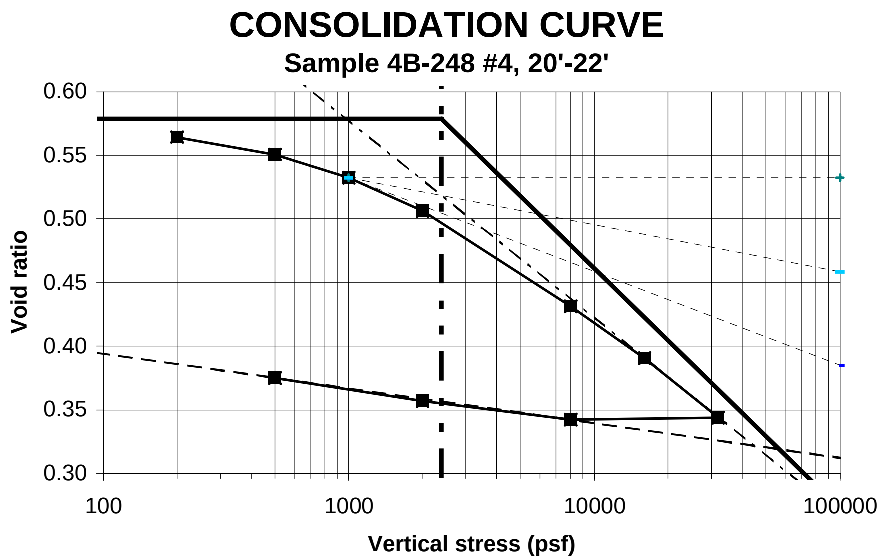
Initial Void Ratio = 0.579
In-Situ Vertical Effective Stress = 2400 psf

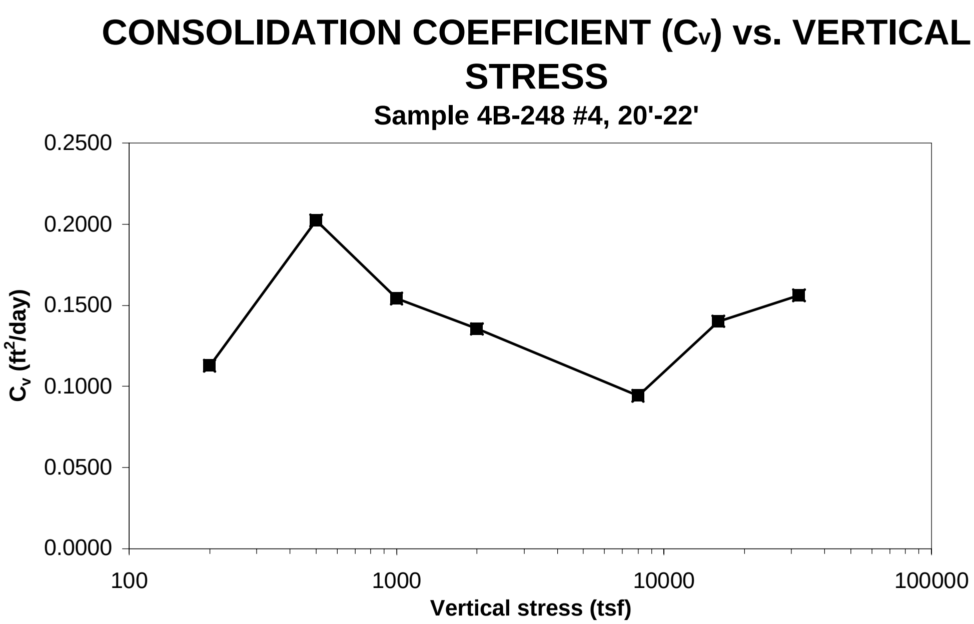
Compression and Swelling Indices

Compression index Cc = 0.155
Field corrected Cc = 0.189
Swelling index Cs = 0.027

Preconsolidation pressure, σ_C

Casagrande Method = 2382 psf
Over-Consolidation Ratio (OCR) = 1.0





Wang Engineering, INC.
Geo-Environmental Engineers
1145 N Main Street
Lombard, IL 60148
Phone : 630 953-9928
Fax: 630 953-9938

Project Name	Kennedy Interchange - Phase 4 - S9070
Source	4W-270 #7 (30'-32')

Cv computation Method: Cassagrande

Initial Void Ratio = 0.805
In-Situ Vertical Effective Stress = 2364 psf

Compression and Swelling Indices

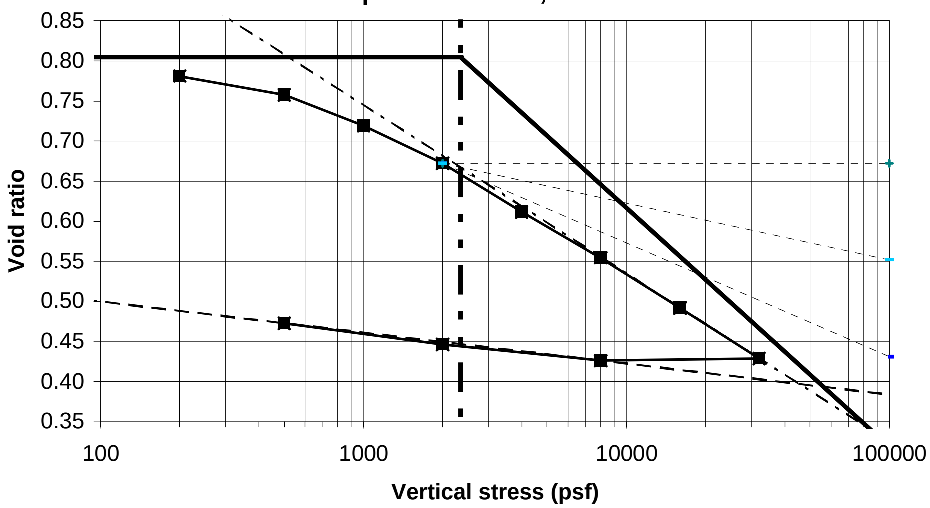
Compression index Cc = 0.210
Field corrected Cc = 0.298
Swelling index Cs = 0.039

Preconsolidation pressure, σ_C

Casagrande Method = 2340 psf
Over-Consolidation Ratio (OCR) = 1.0

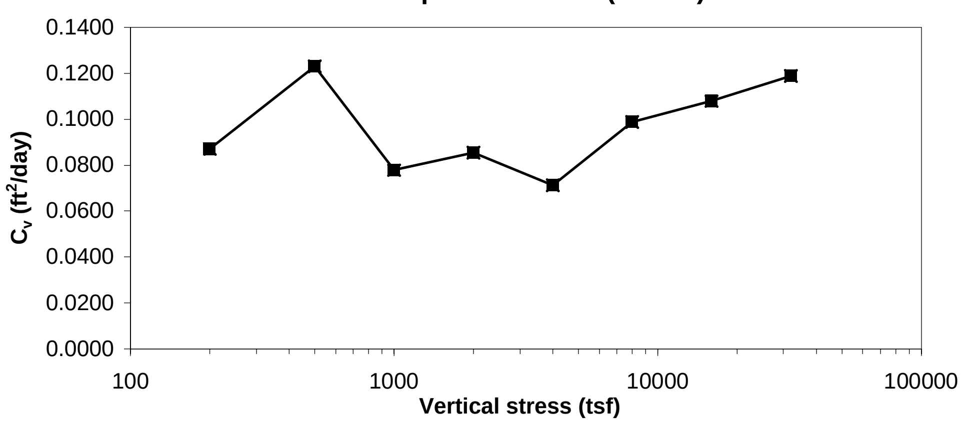
CONSOLIDATION CURVE

Sample 4W-270 #7, 30'-32'



CONSOLIDATION COEFFICIENT (C_v) vs. VERTICAL STRESS

Sample 4W-270 #7 (30'-32')



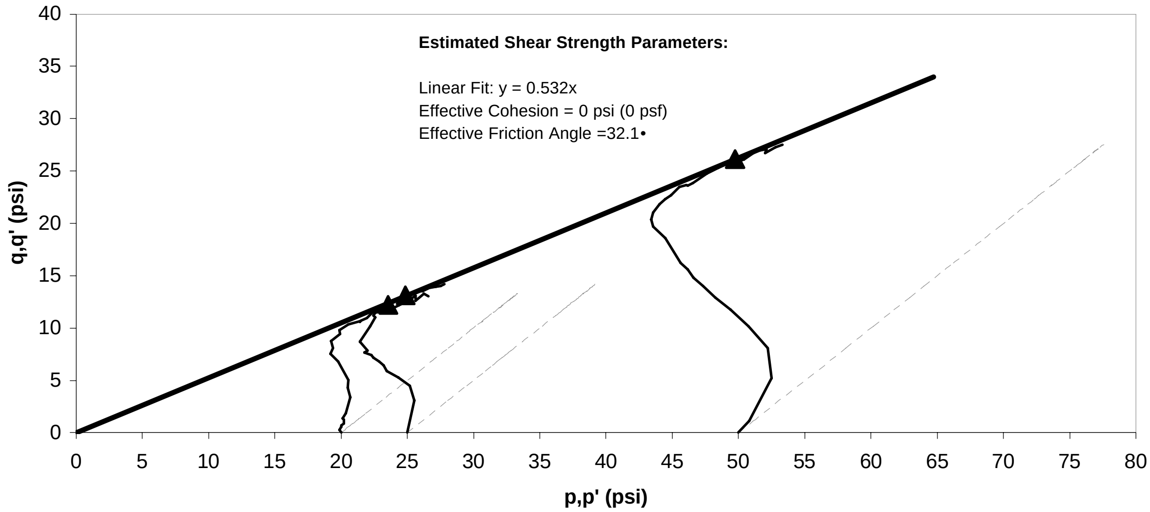
Wang Engineering, INC.
Geo-Environmental Engineers
1145 N Main Street
Lombard, IL 60148
Phone : 630 953-9928
Fax: 630 953-9938

Project: Kennedy Interchange - Phase 4 - S0160: Bridge 64-2

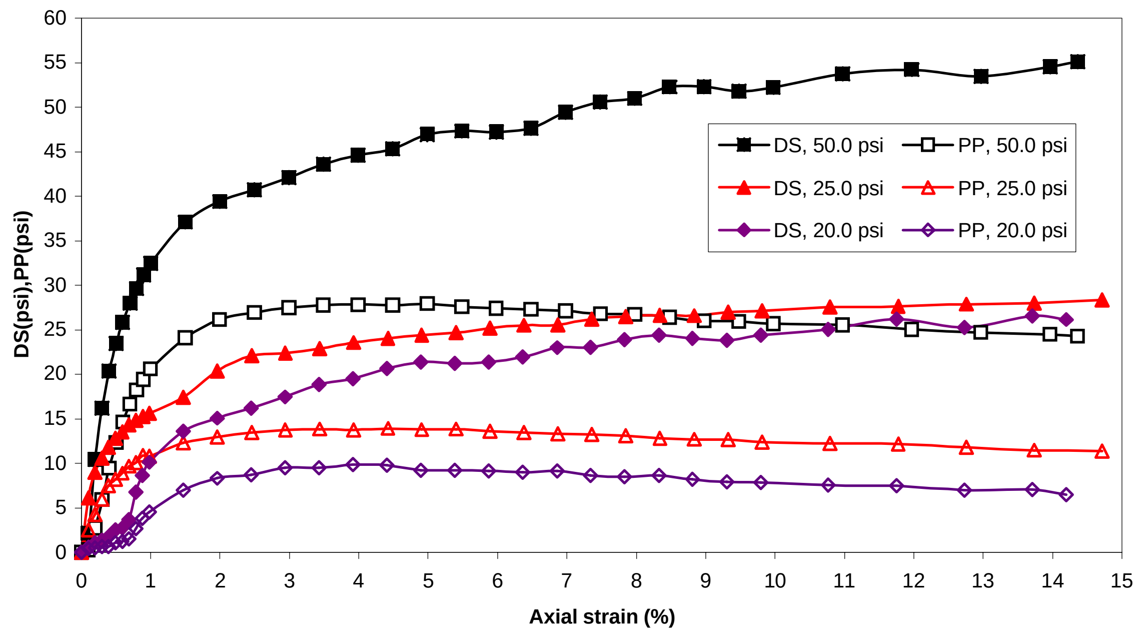
Sample ID: 4B-248/336+ 93,136 Rt., 20'to 27'

Failure Criterion : Maximum Effective Principal Stress Ratio

Total and Effective Stress Paths and Effective Stress Strength Envelope



Deviatoric Stress (DS) and Pore Water Pressure (PP) VS. Axial Strain



APPENDIX C
CORRECTED SPT N-VALUES AND SOIL PARAMETERS

Corrected SPT N-Values
and Soil Parameters
4B-248

Boring 4B-248
Kennedy Interchange Reconstruction - Phase 4
Ramp 42 (3rd ST to I-65 SB)
GWL (extreme) = 0 ft bgs (flood stage)
GWL (normal) = 22.69 ft bgs (Elevation 421 ft) estimated based on Ohio River Pool elevation

Soil Description	Classification (USCS)	Top Elevation (ft)	Bottom Elevation (ft)	Undrained Shear Strength (psf)	Raw Field SPT N_{field} (Blow Count)	Hammer Corrected SPT N_{60} (Blow Count)	Final Corrected SPT $N1_{60}$ (Blow Count)	Internal Friction Angle (degree)
Clayey sandy silt	SC-SM	443.69		125	8	11	13	31
Soft silty clay	CL		434.2	500	14	19	21	
	CL	434.2		250				
	CL			500				
	CL			1000				
	ML		410.7	760				
Silty sand with gravel	SP-SM	410.7			32	43	35	37
	SP-SM				48	64	50	41
	SP-SM				35	47	35	37
	SW-SM		390.7		26	35	25	34
Medium sand	SP	390.7			25	33	24	34
	SP				27	36	25	34
	SP				47	63	42	39
	SP				45	60	39	38
	SP		363.7		39	52	33	36

Equations:
$N1=C_N N_{field}$; $C_N=[0.77\log10(40/\sigma_v')]$, and $C_N<2.0$; [1)]
$N_{60} = (80/60)N_{field}$; [corresponding to 80% hammer efficiency] [1)]
$N1_{60}=C_N N_{60}$; [1)]
$\varphi(deg)=27.1+0.3N_{cor}-0.00054N_{cor}^2$ [2)]

References:

1) AASHTO LRFD Bridge Design Specifications, Interim 2006, Section 10.4.6.2.4
2) Wolff, T. F. (1989). "Pile Capacity Prediction Using Parameter Functions," in *Predicted and Observed Axial Behavior of Piles, Results of a Pile Prediction Symposium*, sponsored by the Geotechnical Engineering Division, ASCE, Evanston, IL, June, 1989, ASCE Geotechnical Special Publication No. 23, pp. 96-106.

Corrected SPT N-Values
and Soil Parameters
4B-249

Boring 4B-249
Kennedy Interchange Reconstruction - Phase 4
Ramp 42 (3rd ST to I-65 SB)
GWL (extreme) = 0 ft bgs (flood stage)
GWL (normal) = 19.58 ft bgs (Elevation 421 ft) estimated based on Ohio River Pool elevation

Soil Description	Classification (USCS)	Top Elevation (ft)	Bottom Elevation (ft)	Undrained Shear Strength (psf)	Raw Field SPT N_{field} (Blow Count)	Hammer Corrected SPT N_{60} (Blow Count)	Final Corrected SPT $N1_{60}$ (Blow Count)	Internal Friction Angle (degree)
Stiff silty clay	SC	440.58		4500	23	31	48	
	CL		432.1	1000	8	11	14	
	CL	432.1		1000	5	7	8	
	CL			750	4	5	5	
	CL-ML			1290				
	CL-ML			230				
	CL-ML		407.6	1250	3	4	3	
Silty sand with gravel	SM	407.6			31	41	34	37
	SM				30	40	32	36
	SM				68	91	70	45
	SM		387.6		35	47	35	37
Medium silty sand	SW-SM	387.6			17	23	16	32
	SW-SM				16	21	15	31
	SP-SM				22	29	20	33
	SP-SM				49	65	43	39
	SP-SM		360.6		33	44	28	35

Equations:

$N1=C_N N_{field}$; $C_N=[0.77\log10(40/\sigma_v')]$, and $C_N<2.0$; [1)]

$N_{60}=(80/60)N_{field}$; [corresponding to 80% hammer efficiency] [1)]

$N1_{60}=C_N N_{60}$; [1)]

$\varphi(deg)=27.1+0.3N_{cor}-0.00054N_{cor}^2$ [2)]

References:

1) AASHTO LRFD Bridge Design Specifications, Interim 2006, Section 10.4.6.2.4

2) Wolff, T. F. (1989). "Pile Capacity Prediction Using Parameter Functions," in *Predicted and Observed Axial Behavior of Piles, Results of a Pile Prediction Symposium*, sponsored by the Geotechnical Engineering Division, ASCE, Evanston, IL, June, 1989, ASCE Geotechnical Special Publication No. 23, pp. 96-106.

Corrected SPT N-Values
and Soi Parameters
4B-250

Boring 4B-250
Kennedy Interchange Reconstruction - Phase 4
Ramp 42 (3rd ST to I-65 SB)
GWL (extreme) = 0 ft bgs (flood stage)
GWL (normal) = 23.07 ft bgs (Elevation 421 ft) estimated based on Ohio River Pool elevation

Soil Description	Classification (USCS)	Top Elevation (ft)	Bottom Elevation (ft)	Undrained Shear Strength (psf)	Raw Field SPT N_{field} (Blow Count)	Hammer Corrected SPT N_{60} (Blow Count)	Final Corrected SPT $N1_{60}$ (Blow Count)	Internal Fric Angle (degree
Silty sand with clay	SC-SM	444.1			6	8	12	31
	SC-SM				8	11	14	31
	SM		433.6		6	8	9	30
Silty clay	CL	433.6		810				
	CL			650				
	CL			1840				
	CL-ML			520				
	CL		408.1	250	34	45	37	
Silty sand with gravel	SP-SM	408.1			34	45	35	37
	SP-SM				34	45	34	37
	SP-SM		391.1		29	39	28	35
Medium sand	SP	391.1			10	13	9	30
	SP				36	48	33	36
	SP		376.1		26	35	23	34
Silty sand with sandy	SW-SM	376.1			37	49	32	36
gravel lenses	SP-SM		see sheet 2 of 2		24	32	20	33

Equations: $N1=C_N N_{field}$; $C_N=[0.77\log10(40/\sigma_v')]$, and $C_N<2.0$; [1)]
$N_{60} = (80/60)N_{field}$; [corresponding to 80% hammer efficiency] [1)]
$N1_{60}=C_N N_{60}$; [1)]
$\varphi(deg)=27.1+0.3N_{cor}-0.00054N_{cor}^2$ [2)]

References: 1) AASHTO LRFD Bridge Design Specifications, Interim 2006, Section 10.4.6.2.4
2) Wolff, T. F. (1989). "Pile Capacity Prediction Using Parameter Functions," in *Predicted and Observed Axial Behavior of Piles, Results of a Pile Prediction Symposium* , sponsored by the Geotechnical Engineering Division, ASCE, Evanston, IL, June, 1989, ASCE Geotechnical Special Publication No. 23, pp. 96-106.

Corrected SPT N-Values
and Soil Parameters
4B-250

Boring 4B-250
Kennedy Interchange Reconstruction - Phase 4
Ramp 42 (3rd ST to I-65 SB)
GWL (extreme) = 0 ft bgs (flood stage)
GWL (normal) = 20.07 ft bgs (Elevation 421 ft) estimated based on Ohio River Pool elevation

Soil Description	Classification (USCS)	Top Elevation (ft)	Bottom Elevation (ft)	Undrained Shear Strength (psf)	Raw Field SPT N_{field} (Blow Count)	Hammer Corrected SPT N_{60} (Blow Count)	Final Corrected SPT $N1_{60}$ (Blow Count)	Internal Fric Angle (degree
Silty sand with sandy	SP-SM				32	43	26	35
gravel lenses	SW-SM				29	39	23	34
	GP-GM				113	151	88	49
	SP-SM				27	36	20	33
	SP-SM				14	19	10	30
	SM				35	47	25	34
	SP-SM				14	19	10	30
	SP-SM		327.6		33	44	23	34

Equations: $N1=C_N N_{field}$; $C_N=[0.77\log10(40/\sigma_v')]$, and $C_N<2.0$; [1)]
$N_{60} = (80/60)N_{field}$; [corresponding to 80% hammer efficiency] [1)]
$N1_{60}=C_N N_{60}$; [1)]
$\varphi(deg)=27.1+0.3N_{cor}-0.00054N_{cor}^2$ [2)]

References:
1) AASHTO LRFD Bridge Design Specifications, Interim 2006, Section 10.4.6.2.4
2) Wolff, T. F. (1989). "Pile Capacity Prediction Using Parameter Functions," in *Predicted and Observed Axial Behavior of Piles, Results of a Pile Prediction Symposium* , sponsored by the Geotechnical Engineering Division, ASCE, Evanston, IL, June, 1989, ASCE Geotechnical Special Publication No. 23, pp. 96-106.

Corrected SPT N-Values
and Soil Parameters
4W-270

Boring 4W-270
Kennedy Interchange Reconstructic
Ramp 42 (3rd ST to I-65 SB)
GWL (extreme) = 0 ft bgs (flood stage)
GWL (normal) = 24.05 ft bgs (Elevation 421 ft) estimated based on Ohio River Pool elevation

Soil Description	Classification (USCS)	Top Elevation (ft)	Bottom Elevation (ft)	Undrained Shear Strength (psf)	Raw Field SPT N_{field} (Blow Count)	Hammer Corrected SPT N_{60} (Blow Count)	Final Corrected SPT $N1_{60}$ (Blow Count)	Internal Friction Angle (degree)
Silty clay	CL	445.05	436.1	3500	23	31	41	
ilty sand w/ grave	SM	436.1	429.1		9	12	14	31
Silty clay	CL	429.1		500	7	9	9	
	CL			125	4	5	5	
	CL			750	8	11	9	
	CL-ML			125				
	CL		405.1	250				
Sandy gravel	GP	405.1			28	37	29	35
	GP				32	43	32	36
	GP		389.6		12	16	12	31
Medium sand	SW-SM	389.6			25	33	23	34
	SP				19	25	17	32
	SP				28	37	25	34
	SP				30	40	26	34
	SP		365.1		24	32	20	33

Equations: $N1=C_N N_{field}$; $C_N=[0.77\log10(40/\sigma_v')]$, and $C_N<2.0$;
$N_{60} = (80/60)N_{field}$; [corresponding to 80% hammer efficiency] [1)]
$N1_{60}=C_N N_{60}$; [1)]
$\varphi(deg)=27.1+0.3N_{cor}-0.00054N_{cor}^2$

References: 1) AASHTO LRFD Bridge Design Specifications, Interim 2006, Section :
2) Wolff, T. F. (1989). "Pile Capacity Prediction Using Parameter Functions," in *Predicted and Observed Axial Behavior of Pile*
a Pile Prediction Symposium , sponsored by the Geotechnical Engineering Division, ASCE, Evanston, IL, June, 1989, ASCE (
Special Publication No. 23, pp. 96-

APPENDIX D
SINGLE STEEL H-PILE CAPACITY EVALUATIONS

SINGLE STEEL H-PILE CAPACITY EVALUATIONS

Resistance Factors for LRFD – Driven Piles

Axial Capacity	
Skin Friction and End Bearing In Clays – Alpha Method	0.35
Skin Friction and End Bearing in Sands – Nordlund/Thurman Method	0.45

Uplift Resistance

Clays	Alpha Method	0.25
Sands	Nordlund Method	0.35

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Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬ò

É¿¬»® Ì¿¾´» ¿¬ Ú´±±¼ Û´»ª¿¬·±²ã ìíçòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Ì·° Þ»¿®·²¹	Î² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»-·-¬¿²½»		¢Î² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Í¬®»²¹¬¸ Ô·³·¬ Í¬¿¬»		¢Î² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Û¨¬®»³» Ûª»²¬ Ô·³·¬ Í¬¿¬»		¢Î² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»-·-¬¿²½» Í¬®»²¹¬¸ Ô·³·¬ Í¬¿¬»	
	øº¬÷	øµ·°-÷	øµ·°-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷
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Ûª¿´«¿¬»¼ Þ¿» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬
É¿¬»® Ì¿¾´» ¿¬ Û´»ª¿¬·±²ã ìíçòð º¬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»·¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»·¬¿²½»		ɸ Î² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»·ª» Î»·¬¿²½» Í¬¿¬·½ ß²¿´§· Ó»¬¸±¼		ɸ Î² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»·ª» Î»·¬¿²½» Ü§²¿³·½ ß²¿´§· Ó»¬¸±¼		ɸ Î² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»·¬¿²½» Í¬¿¬·½ ß²¿´§· Ó»¬¸±¼	
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	éëòðð	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	éêòðð	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	ééòðð	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
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ïî¨ëí ØóÐ·´» øëð µ-·÷

Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬
É¿¬»® Ì¿¾´» ¿¬ Û´»ª¿¬·±² ã ìíçòð º¬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» λ-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î ² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ λ-·-¬¿²½»		ϕ Î ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» λ-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼		ϕ Î ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» λ-·-¬¿²½» Ü§²¿³·½ ß²¿´§-·- Ó»¬¸±¼		ϕ Î ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ λ-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼	
	ø º¬÷	ø µ·°-÷	ø µ·°-÷	ø µ·°-÷	ø ¬±²-÷	ø µ·°-÷	ø ¬±²-÷	ø µ·°-÷	ø ¬±²-÷	ø µ·°-÷	ø ¬±²-÷
Í¿²¼	éçòðð	íçðòîð	ïêòçð	ìðéòïð	îðíòëë	ïéèòçð	èçòìë	îêìòêî	ïíîòíï	ïíîòîé	êêòïì
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	ïïïòðð	êíêòèð	ïçòìð	êëçòíð	íîçòêë	îçîòìð	ïìêòîð	ìîèòëç	îïìòíð	îïèòëè	ïðçòîç

Í¬»»´ ØóÐ·´» Ý¿°¿½·¬·»-
Þí Î Üòï Ð·»®- ïòì
ï î ¨ë í ØóÐ·´» øëð µ-·÷

Û-¬· ³ ¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ì í çòð º¬ò
É¿¬»® Ì¿¾´» ¿¬ Ú´±±¼ Û´»ª¿¬·±²ã ì í çòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò± ³ ·² ¿´ Í ·¼» Î »-·-¬¿²½»	Ò± ³ ·² ¿´ Û²¼ Þ»¿®·² ¹	Î ² Ì ±¬¿´ Ò± ³ ·² ¿´ Ù»±¬»½¸ ² ·½¿´ ß¨·¿´ Î »-·-¬¿²½»		ϕ Î ² Ì ±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸ ² ·½¿´ ß¨·¿´ Ý± ³ °®»--·ª» Î »-·-¬¿²½» Í ¬¿¬·½ ß ²¿´§-·- Ó»¬¸ ±¼		ϕ Î ² Ì ±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸ ² ·½¿´ ß¨·¿´ Ý± ³ °®»--·ª» Î »-·-¬¿²½» Ü§²¿ ³ ·½ ß ²¿´§-·- Ó»¬¸ ±¼		ϕ Î ² Ì ±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸ ² ·½¿´ ß¨·¿´ Ë°´·º¬ Î »-·-¬¿²½» Í ¬¿¬·½ ß ²¿´§-·- Ó»¬¸ ±¼	
	øº¬÷	øµ·° -÷	øµ·° -÷	øµ·° -÷	ø¬±² -÷	øµ·° -÷	ø¬±² -÷	øµ·° -÷	ø¬±² -÷	øµ·° -÷	ø¬±² -÷
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	ï ï íòðð	êë îòîð	ïçòì ð	êéïòêð	í í ëòèð	îçéòçî	ï ìèòçê	ì íêòëì	î ï èòîé	î î íòçé	ï ï ïòçç
	ï ï ìòðð	êëçòçð	ïçòì ð	êéçòíð	í í çòêë	íðïòíç	ïëðòêç	ì ì ïòëë	î î ðòéé	î îêòêé	ï ï íòíí
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Í¬»»´ ØòÐò´» Ý¿°¿½·¬»-
ÞÍ ÎÜòï Ð·»®- ïóì
ïì¨éí ØòÐò´» øëð µ-·÷

Û-¬· ³ ¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬
Ê¿¬»® Ì¿¾´» ¿¬ Û´»ª¿¬·±² ã ìíçòð º¬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»-·-¬¿²½»		ϕÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼		ϕÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Ü§²¿³·½ ß²¿´§-·- Ó»¬¸±¼		ϕÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼	
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Í¬»»´ ØóÐ·´» Ý¿°¿½·¬·»-
Þí Î Üóï Ð·»®- ïóì
ïì¨éí ØóÐ·´» øëð µ-·÷

Û-¬· ³ ¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬
Ê¿»®¿¹» Ì¿¾´» ¿¬ Û´±±¼ Û´»ª¿¬·±² ã ìíçòð º¬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»--·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»--·-¬¿²½»		ϕÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»--·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼		ϕÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»--·-¬¿²½» Ü§²¿³·½ ß²¿´§-·- Ó»¬¸±¼		ϕÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»--·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼	
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Í¬»»´ Øò Ð·´» Ý¿°¿½·¬·»-
Þí Î Üòï Ð·»®- ïòì
ïì¨éí Øò Ð·´» øëð µ-÷

Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬
É¿¬»® Ì¿¾´» ¿¬ Ú´±±¼ Û´»ª¿¬·±² ã ìíçòð º¬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î ² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»-·-¬¿²½»		ϕÎ ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼		ϕÎ ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Ü§²¿³·½ ß²¿´§-·- Ó»¬¸±¼		ϕÎ ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼	
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Í¬»»´ ØÐ Ð·´» Ý¿°¿½·¬·»-
ÞÍ Î Üóï Ð·»®- ïóì
ïì¨èç ØÐ Ð·´» øëð µ-·÷

Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬
Ù®±«²¼ Í«®º¿½» Û´»ª¿¬·±² ã ìíçòð º¬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»-·-¬¿²½»		ϕÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼		ϕÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Ü§²¿³·½ ß²¿´§-·- Ó»¬¸±¼		ϕÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼	
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Í¬»»´ Ø·Ð·´» Ý¿°¿½·¬·»-
Þí Î Ü·ï Ð·»®- ïòì
ïì ¨èç Ø·Ð·´» øëð µ-·÷

Û¬ ³ ¿·»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ì íçòð ºí
É¿»® Ì¿¾´» ¿¬ Û´»ª¿¬·±² ã ì íçòð ºí

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î ₂ Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»-·-¬¿²½»		ϕÎ ₂ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼		ϕÎ ₂ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Ü§²¿³·½ ß²¿´§-·- Ó»¬¸±¼		ϕÎ ₂ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼	
	øº¬÷	øµ·°-÷	øµ·°-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷
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Í¬»»´ ØòÐ·´» Ý¿°¿½·¬»-
Þí Î Üòï Ð·®- ïòì
ï ì ¨èç ØòÐ·´» ø ëð µ-·÷

Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬ò
É¿¬»® Ì¿¾´» ¿¬ Û´±±¼ Û´»ª¿¬·±² ã ìíçòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»-·-¬¿²½»		φÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼		φÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½» Ü§²¿³·½ ß²¿´§-·- Ó»¬¸±¼		φÎ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»-·-¬¿²½» Í¬¿¬·½ ß²¿´§-·- Ó»¬¸±¼	
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Í¬»»´ ØóÐ·´» Ü®·ª·²¹ Î»·¬¿²½»
Þí Î Üóï Ð·»® ïóì
ïî¨ëí ØóÐ·´» øëð µ·÷

Û¬· ³ ¿¬»¼ Þ¿» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬ò
É¿¬»® Ì¿¾´» Ò»¿® Ò±® ³ ¿´ Ð±±´ Û´»ª¿¬·±²ã ìîïòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò± ³ ·² ¿´ Í ·¼» Î »·¬¿²½»	Ò± ³ ·² ¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î »·¬¿²½»	
	øº¬÷	øµ·° ÷	øµ·° ÷	øµ·° ÷	ø¬±² ÷
Í¿²¼	ðòðð	ðòðð	ðòìð	ðòìð	ðòîð
	ïòðð	îòèð	îòðð	ìòïð	îòðë
	îòðð	íòèð	îòîð	ëòðë	îòëí
	íòðð	ìòçð	ïòçð	ëòíð	îòêë
	ìòðð	êòîð	ïòîð	ëòîë	îòêí
	ëòðð	éòéð	ðòëð	ëòíð	îòêë
Ý´¿§	êòðð	çòëð	ðòëð	êòîð	íòïð
	éòðð	ïïòìð	ðòëð	éòïë	íòëè
	èòðð	ïíòìð	ðòëð	èòïë	ìòðè
	çòðð	ïëòìð	ðòëð	çòïë	ìòëè
	ïðòðð	ïéòìð	ðòëð	ïðòïë	ëòðè
	ïïòðð	ïçòìð	ðòëð	ïïòïë	ëòëè
	ïîòðð	îïòìð	ðòëð	ïîòïë	êòðè
	ïíòðð	îíòìð	ðòëð	ïíòïë	êòëè
	ïìòðð	îëòíð	ðòëð	ïìòïð	éòðë
	ïëòðð	îéòíð	ðòëð	ïëòïð	éòëë
	ïêòðð	îçòíð	ðòëð	ïêòïð	èòðë
	ïéòðð	íïòíð	ðòëð	ïéòêð	èòèð
	ïèòðð	ííòíð	ðòëð	ïçòïð	çòëë
	ïçòðð	íëòíð	ðòëð	îðòêð	ïðòíð
	îðòðð	íéòîð	ðòëð	îîòðí	ïïòðï
	îïòðð	íçòîð	ðòëð	îíòëí	ïïòéê
	îîòðð	ìïòîð	ðòëð	îëòðí	ïîòëï
	îíòðð	ìíòîð	ðòëð	îêòðí	ïíòðï
	îìòðð	ìëòîð	ðòëð	îéòðí	ïíòëï
	îëòðð	ìéòîð	ðòëð	îèòðí	ïìòðï
	îêòðð	ìçòîð	ðòëð	îçòðí	ïìòëï
	îéòðð	ëïòïð	ðòëð	îçòçè	ïìòçç
	îèòðð	ëíòïð	ðòëð	íðòçè	ïëòìç
	îçòðð	ëëòïð	îòðð	ííòìè	ïêòéì
	íðòðð	ëéòïð	ëòðð	íéòìè	ïèòéì
	íïòðð	ëçòïð	éòçð	ìïòíè	îðòêç

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò± ³ ·² ¿´ Í ·¼» Î »·¬¿²½»	Ò± ³ ·² ¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î »·¬¿²½»	
	øº¬÷	øµ·° ÷	øµ·° ÷	øµ·° ÷	ø¬±² ÷
Í¿²¼	íîòðð	êíòïð	éòçð	ìíòíè	îïòêç
	ííòðð	êçòïð	éòçð	ìéòèè	îíòçì
	íìòðð	ééòîð	éòçð	ëîòìë	îêòîí
	íëòðð	èïòìð	éòçð	ëéòïð	îèòëë
	íêòðð	èéòèð	éòçð	êïòçð	íðòçë
	íéòðð	çìòîð	éòçð	êêòéð	ííòíë
	íèòðð	ïððòéð	éòçð	éïòëè	íëòéç
	íçòðð	ïðéòíð	éòçð	éêòëí	íèòîê
	ìðòðð	ïïìòðð	éòçð	èïòëë	ìðòéè
	ìïòðð	ïîðòèð	éòçð	èêòêë	ìíòíí
	ìîòðð	ïîéòéð	éòçð	çïòèí	ìëòçï
	ìíòðð	ïíìòéð	éòçð	çéòðè	ìèòëì
	ììòðð	ïìïòèð	éòçð	ïðîòìð	ëïòîð
	ìëòðð	ïìçòðð	éòçð	ïðéòèð	ëíòçð
	ìêòðð	ïëêòíð	éòçð	ïïíòîè	ëêòêì
	ìéòðð	ïêíòéð	éòçð	ïïèòèí	ëçòìï
	ìèòðð	ïéïòîð	éòçð	ïîìòìë	êîòîí
	ìçòðð	ïéèòèð	éòçð	ïíðòïë	êëòðè
	ëðòðð	ïèêòëð	éòçð	ïíëòçí	êéòçê
	ëïòðð	ïçìòíð	éòçð	ïìïòéè	éðòèç
	ëîòðð	îðîòîð	éòçð	ïìéòéð	éíòèë
	ëíòðð	îïðòîð	éòçð	ïëíòéð	éêòèë
	ëìòðð	îïèòíð	éòçð	ïëçòéè	éçòèç
	ëëòðð	îîêòìð	éòçð	ïêëòèë	èîòçí
	ëêòðð	îíìòéð	éòçð	ïéîòðè	èêòðì
	ëéòðð	îìíòíð	éòçð	ïéèòíè	èçòïç
	ëèòðð	îëïòêð	éòçð	ïèìòéë	çîòíè
	ëçòðð	îêðòîð	éòçð	ïçïòîð	çëòêð
	êðòðð	îêèòèð	éòçð	ïçéòêë	çèòèí
	êïòðð	îééòêð	éòçð	îðìòîë	ïðîòïí
	êîòðð	îèêòëð	éòçð	îïðòçí	ïðëòìê
	êíòðð	îçëòìð	éòçð	îïéòéê	ïðèòèð

Í¬»»´ ØóÐ·´» Ü®·ª»² Î»·¬¿²½»
Þí Î Üóï Ð·»®- ïóì
ï î¨ëí ØóÐ·´» øëð µ-·÷

Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ì íçòð º¬ò
É¿¬»® Ì¿¾´» Ò»¿® Ò±®³¿´ Ð±±´ Û´»ª¿¬·±²ã ì î ïòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î»-·-¬¿²½»	
	ø º¬÷	øµ·° -÷	øµ·° -÷	øµ·° -÷	ø¬±² -÷
Í¿²¼	êìòðð	íðìòëð	éòçð	îîìòìí	ïïîòîï
	êëòðð	íïíòéð	éòçð	îíïòíí	ïïëòêê
	êêòðð	íîîòçð	éòçð	îíèòîí	ïïçòïï
	êéòðð	ííîòíð	éòçð	îìëòîè	ïîîòêì
	êèòðð	íìïòèð	éòçð	îëîòìð	ïîêòîð
	êçòðð	íëïòíð	éòçð	îëçòëí	ïîçòéê
	éðòðð	íêïòðð	éòçð	îêêòèð	ïíííòìð
	éïòðð	íéðòéð	éòçð	îéìòðè	ïíéòðì
	éîòðð	íèðòêð	éòçð	îèïòëð	ïìðòéë
	éíòðð	íçðòëð	éòçð	îèèòçí	ïììòìê
	éìòðð	ìððòêð	éòçð	îçêòëð	ïìèòîë
	éëòðð	ìïðòéð	éòçð	íðìòðè	ïëîòðì
	éêòðð	ìîïòðð	éòçð	íïïòèð	ïëëòçð
	ééòðð	ìíïòíð	éòçð	íïçòëí	ïëçòéê
	éèòðð	ììïòèð	éòçð	íîéòìð	ïêíòéð
	éçòðð	ìëîòíð	éòçð	ííëòîè	ïêéòêì
	èðòðð	ìêîòçð	éòçð	ìííòîí	ïéïòêï
	èïòðð	ìéíòéð	éòçð	íëïòíí	ïéëòêê
	èîòðð	ìèìòëð	éòçð	íëçòìí	ïéçòéï
	èíòðð	ìçëòìð	éòçð	íêéòêð	ïèíòèð
	èìòðð	ëðêòëð	éòçð	íéëòçí	ïèéòçê
	èëòðð	ëïéòêð	éòçð	íèìòîë	ïçîòïí
	èêòðð	ëîèòèð	éòçð	íçîòêë	ïçêòíí
	èéòðð	ëìðòîð	éòçð	ìðïòîð	îððòêð
	èèòðð	ëëïòêð	éòçð	ìðçòéë	îðìòèè
	èçòðð	ëêíòïð	éòçð	ìïèòíè	îðçòïç
	çðòðð	ëéìòéð	éòçð	ìîéòðè	îïíòëì
	çïòðð	ëèêòìð	éòçð	ìíëòèë	îïéòçí
	çîòðð	ëçèòíð	éòçð	ììììòéè	îîîòíç
	çíòðð	êïðòîð	éòçð	ìëíòéð	îîêòèë
	çìòðð	êîîòîð	éòçð	ìêîòéð	îíïòíë
	çëòðð	êíìòíð	éòçð	ìéïòéè	îíëòèç

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î»-·-¬¿²½»	
	ø º¬÷	øµ·° -÷	øµ·° -÷	øµ·° -÷	ø¬±² -÷
Í¿²¼	çêòðð	êìêòëð	éòçð	ìèðòçí	îìðòìê
	çéòðð	êëèòèð	éòçð	ìçðòïë	îìëòðè
	çèòðð	êéïòîð	éòçð	ìççòìë	îìçòéí
	ççòðð	êèíòéð	éòçð	ëðèòèí	îëìòìï
	ïððòðð	êçêòíð	éòçð	ëïèòîè	îëçòïì
	ïðïòðð	éðçòðð	éòçð	ëîéòèð	îêíòçð
	ïðîòðð	éîïòèð	éòçð	ëíéòìð	îêèòéð
	ïðíòðð	éíìòéð	éòçð	ëìéòðè	îéíòëì
	ïðìòðð	éìéòéð	éòçð	ëëêòèí	îéèòìï
	ïðëòðð	éêðòèð	éòçð	ëêêòêë	îèíòíí
	ïðêòðð	ééìòðð	éòçð	ëéêòëë	îèèòîè
	ïðéòðð	éèéòíð	éòçð	ëèêòëí	îçíòîê
	ïðèòðð	èððòéð	éòçð	ëçêòëè	îçèòîç
	ïðçòðð	èïìòîð	éòçð	êðêòéð	íðíòíë
	ïïðòðð	èîéòéð	éòçð	êïêòèí	íðèòìï
	ïïïòðð	èìïòìð	éòçð	êîéòïð	íïíòëë
	ïïîòðð	èëëòîð	éòçð	êíéòìë	íïèòéí
	ïïíòðð	èêçòïð	éòçð	êìéòèè	íîíòçì
	ïïìòðð	èèíòïð	éòçð	êëèòíè	íîçòïç
	ïïëòðð	èçéòïð	éòçð	êêèòèè	ííìòìì

Í¬»»´ ØóÐ·´» Ü®·ª·²¹ Î»·¬¿²½»
Þí Îܳï Ð·»® ïóì
ïì¨éí ØóÐ·´» øëð µ·÷

Û¬·³¿¬»¼ Þ¿» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬
Ê¿®» Ì¿¾´» Ò»¿® Ò±®³¿´ Ð±±´ Û´»ª¿¬·±²ã ìîïòð º¬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»·¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î»·¬¿²½»	
	øº¬÷	øµ·°÷	øµ·°÷	øµ·°÷	ø¬±²÷
Í¿²¼	ðòðð	ðòðð	ðòêð	ðòêð	ðòíð
	ïòðð	íòíð	îòèð	ëòîè	îòêì
	îòðð	ìòëð	íòðð	êòíè	íòïç
	íòðð	ëòèð	îòíð	êòíí	íòïê
	ìòðð	éòìð	ïòëð	êòíí	íòïê
	ëòðð	çòïð	ðòéð	êòíè	íòïç
Ý´¿§	êòðð	ïïòîð	ðòéð	éòìí	íòéï
	éòðð	ïíòëð	ðòéð	èòëè	ìòîç
	èòðð	ïëòçð	ðòéð	çòéè	ìòèç
	çòðð	ïèòîð	ðòéð	ïðòçí	ëòìê
	ïðòðð	îðòêð	ðòéð	ïîòïí	êòðê
	ïïòðð	îîòçð	ðòéð	ïíòîè	êòêì
	ïîòðð	îëòíð	ðòéð	ïìòìè	éòîì
	ïíòðð	îéòêð	ðòéð	ïëòêí	éòèï
	ïìòðð	íðòðð	ðòéð	ïêòèí	èòìï
	ïëòðð	íîòíð	ðòéð	ïéòçè	èòçç
	ïêòðð	íìòéð	ðòéð	ïçòïè	çòëç
	ïéòðð	íéòðð	ðòéð	îðòçð	ïðòìë
	ïèòðð	íçòìð	ðòéð	îîòéð	ïïòíë
	ïçòðð	ìïòéð	ðòéð	îìòìí	ïîòîï
	îðòðð	ììòïð	ðòéð	îêòîí	ïíòïï
	îïòðð	ìêòìð	ðòéð	îéòçë	ïíòçè
	îîòðð	ìèòèð	ðòéð	îçòéë	ïìòèè
	îíòðð	ëïòïð	ðòéð	íðòçð	ïëòìë
	îìòðð	ëíòëð	ðòéð	íîòïð	ïêòðë
	îëòðð	ëëòèð	ðòéð	ííòîë	ïêòêí
	îêòðð	ëèòîð	ðòéð	ííòìë	ïéòîí
	îéòðð	êðòëð	ðòéð	íëòêð	ïéòèð
	îèòðð	êîòçð	ðòéð	íêòèð	ïèòìð
	îçòðð	êëòîð	ìòïð	ììòíë	îðòêè
	íðòðð	êéòêð	éòëð	ìëòçë	îîòçè

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»·¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î»·¬¿²½»	
	øº¬÷	øµ·°÷	øµ·°÷	øµ·°÷	ø¬±²÷
Ý´¿§	íïòðð	êçòçð	ïðòçð	ëðòëð	îëòîë
Í¿²¼	íîòðð	éìòêð	ïðòçð	ëîòèë	îêòìí
	ííòðð	èïòèð	ïðòçð	ëèòîë	îçòïí
	íìòðð	èçòðð	ïðòçð	êíòêë	íïòèí
	íëòðð	çêòìð	ïðòçð	êçòîð	íìòêð
	íêòðð	ïðíòèð	ïðòçð	éìòéë	íéòíè
	íéòðð	ïïïòìð	ïðòçð	èðòìë	ìðòîí
	íèòðð	ïïçòïð	ïðòçð	èêòîí	ìíòïï
	íçòðð	ïîéòðð	ïðòçð	çîòïë	ìêòðè
	ìðòðð	ïíìòçð	ïðòçð	çèòðè	ìçòðì
	ìïòðð	ïìíòðð	ïðòçð	ïðìòïë	ëîòðè
	ìîòðð	ïëïòïð	ïðòçð	ïïðòîí	ëëòïï
	ìíòðð	ïëçòìð	ïðòçð	ïïêòìë	ëèòîí
	ììòðð	ïêéòèð	ïðòçð	ïîîòéë	êïòíè
	ìëòðð	ïéêòìð	ïðòçð	ïîçòîð	êìòêð
	ìêòðð	ïèëòðð	ïðòçð	ïíëòêë	êéòèí
	ìéòðð	ïçíòèð	ïðòçð	ïìîòîë	éïòïí
	ìèòðð	îðîòêð	ïðòçð	ïìèòèë	éìòìí
	ìçòðð	îïïòêð	ïðòçð	ïëëòêð	ééòèð
	ëðòðð	îîðòéð	ïðòçð	ïêîòìí	èïòîï
	ëïòðð	îîçòçð	ïðòçð	ïêçòíí	èìòêê
	ëîòðð	îíçòíð	ïðòçð	ïéêòíè	èèòïç
	ëíòðð	îìèòéð	ïðòçð	ïèíòìí	çïòéï
	ëìòðð	îëèòíð	ïðòçð	ïçðòêí	çëòíï
	ëëòðð	îêèòðð	ïðòçð	ïçéòçð	çèòçë
	ëêòðð	îééòèð	ïðòçð	îðëòîë	ïðîòêí
	ëéòðð	îèéòéð	ïðòçð	îïîòêè	ïðêòíì
	ëèòðð	îçéòéð	ïðòçð	îîðòïè	ïïðòðç
	ëçòðð	íðéòçð	ïðòçð	îîéòèí	ïïíòçï
	êðòðð	íïèòïð	ïðòçð	îíëòìè	ïïéòéì
	êïòðð	ííèòëð	ïðòçð	îìíòîè	ïîïòêì

Í¬»»´ ØóÐ·´» Ü®·ª¸²¹ Î»·¬¿²½»
Þí Î Üóï Ð·»®- ïóì
ïì¨éí ØóÐ·´» øëð µ·÷

Û¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬
É¿¬»® Ì¿¾´» Ò»¿® Ò±®³¿´ Ð±±´ Û´»ª¿¬·±²ã ììïòð º¬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»·¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î»·¬¿²½»	
	øº¬÷	øµ·°-÷	øµ·°-÷	øµ·°-÷	ø¬±²-÷
Í¿²¼	êîòðð	ííçòðð	ïðòçð	îëïòïë	ïîëòëè
	êíòðð	íìçòêð	ïðòçð	îëçòïð	ïîçòëë
	êìòðð	íêðòìð	ïðòçð	îêéòîð	ïííòêð
	êëòðð	íéïòîð	ïðòçð	îéëòíð	ïíéòêë
	êêòðð	íèîòîð	ïðòçð	îèíòëë	ïìïòéè
	êéòðð	íçíòíð	ïðòçð	îçïòèè	ïìëòçì
	êèòðð	ìðìòëð	ïðòçð	íððòîè	ïëðòïì
	êçòðð	ìïëòèð	ïðòçð	íðèòéë	ïëìòíè
	éðòðð	ìîéòîð	ïðòçð	íïéòíð	ïëèòêë
	éïòðð	ìíèòéð	ïðòçð	íîëòçí	ïêîòçê
	éîòðð	ìëðòìð	ïðòçð	ííìòéð	ïêéòíë
	éíòðð	ìêîòîð	ïðòçð	íìíòëë	ïéïòéè
	éìòðð	ìéìòïð	ïðòçð	íëîòìè	ïéêòîì
	éëòðð	ìèêòïð	ïðòçð	íêïòìè	ïèðòéì
	éêòðð	ìçèòîð	ïðòçð	íéðòëë	ïèëòîè
	ééòðð	ëïðòìð	ïðòçð	íéçòéð	ïèçòèë
	éèòðð	ëîîòèð	ïðòçð	íèçòðð	ïçìòëð
	éçòðð	ëíëòîð	ïðòçð	íçèòíè	ïççòïç
	èðòðð	ëìéòçð	ïðòçð	ìðéòèí	îðíòçï
	èïòðð	ëêðòêð	ïðòçð	ìïéòíë	îðèòêè
	èîòðð	ëéíòìð	ïðòçð	ìîêòçë	îïíòìè
	èíòðð	ëèêòîð	ïðòçð	ìíêòêí	îïèòíï
	èìòðð	ëççòìð	ïðòçð	ììêòìë	îîíòîí
	èëòðð	êïîòëð	ïðòçð	ìëêòîè	îîèòïì
	èêòðð	êîëòèð	ïðòçð	ìêêòîë	îííòïí
	èéòðð	êíçòîð	ïðòçð	ìéêòíð	îíèòïë
	èèòðð	êëîòèð	ïðòçð	ìèêòëð	îìíòîë
	èçòðð	êêêòìð	ïðòçð	ìçêòéð	îìèòíë
	çðòðð	êèðòïð	ïðòçð	ëðêòçè	îëíòìç
	çïòðð	êçìòðð	ïðòçð	ëïéòìð	îëèòéð
	çîòðð	éðèòðð	ïðòçð	ëîéòçð	îêíòçë

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»·¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î»·¬¿²½»	
	øº¬÷	øµ·°-÷	øµ·°-÷	øµ·°-÷	ø¬±²-÷
Í¿²¼	çíòðð	éîîòïð	ïðòçð	ëíèòìè	îêçòîì
	çìòðð	éíêòíð	ïðòçð	ëìçòïë	îéìòëé
	çëòðð	éëðòêð	ïðòçð	ëëçòèë	îéçòçí
	çêòðð	éêëòïð	ïðòçð	ëéðòéí	îèëòíê
	çéòðð	éçéòéð	ïðòçð	ëèïòêè	îçðòèì
	çèòðð	éçìòïð	ïðòçð	ëçîòêí	îçêòíï
	ççòðð	èðçòïð	ïðòçð	êðíòéí	íðïòèê
	ïððòðð	èîìòðð	ïðòçð	êïìòçð	íðéòìë
	ïðïòðð	èíçòïð	ïðòçð	êîêòîí	íïíòïï
	ïðîòðð	èëìòîð	ïðòçð	êíéòëë	íïèòéè
	ïðíòðð	èêçòëð	ïðòçð	êìçòðí	íîìòëï
	ïðìòðð	èèìòçð	ïðòçð	êêðòëè	íîðòîç
	ïðëòðð	çððòìð	ïðòçð	êéîòîð	íîêòïð
	ïðêòðð	çïêòðð	ïðòçð	êèíòçð	íìïòçë
	ïðéòðð	çíïòéð	ïðòçð	êçëòêè	íìéòèì
	ïðèòðð	çìéòëð	ïðòçð	éðéòëí	íëíòéê
	ïðçòðð	çêíòëð	ïðòçð	éïçòëí	íëçòéê
	ïïðòðð	çéçòêð	ïðòçð	éíïòêð	íêëòèð
	ïïïòðð	ççëòèð	ïðòçð	éìíòéë	íéïòèè
	ïïîòðð	ïðïîòïð	ïðòçð	éëêòçè	íéèòçç
	ïïíòðð	ïðîèòëð	ïðòçð	éêèòîè	íèìòïì
	ïïìòðð	ïðìëòððð	ïðòçð	éèðòêë	íçðòíí
	ïïëòðð	ïðêïòéð	ïðòçð	éçíòïè	íçêòëç

Í¬»»´ ØóÐ·´» Ü®·ª»² Î»·¬¿²½»
Þí Î Üòï Ð·»®- ïóì
ïì¨èç ØóÐ·´» øëðµ÷

Û¬·³¿¬»¼ Þ¿» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬ò
É¿¬»® Ì¿¾´» Ò»¿® Ò±®³¿´ Ð±±´ Û´»ª¿¬·±²ã ìîïòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»·¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î»·¬¿²½»	
	ø º¬÷	øµ·° ÷	øµ·° ÷	øµ·° ÷	ø¬±² ÷
	ðòðð	ðòðð	ðòéð	ðòéð	ðòíë
	ïòðð	íòíð	íòëð	ëòçè	îòçç
	îòðð	ìòëð	íòéð	éòðè	íòëì
Í¿²¼	íòðð	ëòçð	îòèð	êòèè	íòìì
	ìòðð	éòëð	ïòèð	êòêè	íòíì
	ëòðð	çòîð	ðòèð	êòëí	íòîê
	êòðð	ïïòíð	ðòèð	éòëè	íòéç
	éòðð	ïíòéð	ðòèð	èòéè	ìòíç
	èòðð	ïêòïð	ðòèð	çòçè	ìòçç
	çòðð	ïèòìð	ðòèð	ïïòïí	ëòëê
	ïðòðð	îðòèð	ðòèð	ïîòíí	êòïê
	ïïòðð	îíòîð	ðòèð	ïíòëí	êòéê
	ïîòðð	îëòêð	ðòèð	ïìòéí	éòíê
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	ïêòðð	íëòïð	ðòèð	ïçòìè	çòéì
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Ý´¿§	ïèòðð	íçòèð	ðòèð	îíòðð	ïïòëð
	ïçòðð	ìîòîð	ðòèð	îìòèð	ïîòìð
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	îìòðð	ëìòïð	ðòèð	íîòéë	ïêòíè
	îëòðð	ëêòëð	ðòèð	ííòéë	ïêòèè
	îêòðð	ëèòçð	ðòèð	íìòçë	ïéòìè
	îéòðð	êïòîð	ðòèð	íêòïð	ïèòðë
	îèòðð	êíòêð	ðòçð	íéòìð	ïèòéð
	îçòðð	êêòðð	ëòïð	íïòíð	îïòìð
	íðòðð	êèòìð	çòîð	ìèòïð	îìòðë

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»·¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î»·¬¿²½»	
	ø º¬÷	øµ·° ÷	øµ·° ÷	øµ·° ÷	ø¬±² ÷
Ý´¿§	íïòðð	éðòéð	ïíòïð	ëíòíë	îêòêè
	íîòðð	éíòïð	ïíòïð	ëëòéë	îéòèè
	ííòðð	èîòéð	ïíòïð	êïòïë	íðòëè
	íìòðð	çðòðð	ïíòïð	êêòêí	ííòíï
	íëòðð	çéòíð	ïíòïð	éîòîë	íêòïí
	íêòðð	ïðëòïð	ïíòïð	ééòçë	íèòçè
	íéòðð	ïïîòéð	ïíòïð	èíòêë	ìïòèí
	íèòðð	ïîðòíð	ïíòïð	èçòëð	ììòéë
	íçòðð	ïîèòíð	ïíòïð	çëòëð	ìéòéë
	ìðòðð	ïíêòíð	ïíòïð	ïðïòëð	ëðòéë
	ìïòðð	ïììòêð	ïíòïð	ïðéòëè	ëíòéç
	ìîòðð	ïëîòçð	ïíòïð	ïïíòèð	ëêòçð
	ìíòðð	ïêïòíð	ïíòïð	ïîðòïð	êðòðë
	ììòðð	ïêçòèð	ïíòïð	ïîêòìè	êíòîì
	ìëòðð	ïéèòìð	ïíòïð	ïííòçí	êêòçê
Í¿²¼	ìêòðð	ïèéòîð	ïíòïð	ïìïòèí	éðòçï
	ìéòðð	ïçêòðð	ïíòïð	ïìèòíí	éìòïê
	ìèòðð	îðëòïð	ïíòïð	ïëìòéè	éçòèé
	ìçòðð	îïìòìð	ïíòïð	ïêïòðì	èðòëì
	ëðòðð	îîíòçð	ïíòïð	ïêéòëè	èíòìï
	ëïòðð	îííòêð	ïíòïð	ïéìòëè	èêòèì
	ëîòðð	îìíòêð	ïíòïð	ïèìòèð	çïòèé
	ëíòðð	îëííòïð	ïíòïð	ïèçòìí	çìòéï
	ëìòðð	îêíòçð	ïíòïð	ïçìòíì	çéòìè
	ëëòðð	îéìòïð	ïíòïð	îðîòîë	ïðïòïí
	ëêòðð	îèìòëð	ïíòïð	îðçòíð	ïðìòêë
	ëéòðð	îçëòïð	ïíòïð	îïéòîð	ïðèòìì
	ëèòðð	íðêòïð	ïíòïð	îîëòêð	ïïîòèï
	ëçòðð	íïéòïð	ïíòïð	îííòéé	ïïêòèè
	êðòðð	íîèòìð	ïíòïð	îìïòçì	ïîðòçé
	êïòðð	ííçòëð	ïíòïð	îìèòéí	ïîìòîë

Í¬»»´ ØóÐ·´» Ü®·ª¸²¹ Î»--·-¬¿²½»
Þí Î Üóï Ð·»®- ïóì
ïì¨èç ØóÐ·´» øëðµ-·÷

Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬ò
É¿¬»® Ì¿¾´» Ò»¿® Ò±®³¿´ Ð±±´ Û´»ª¿¬·±²ã ìîïòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î»-·-¬¿²½»	
	øº¬÷	øµ·°-÷	øµ·°-÷	øµ·°-÷	ø¬±²-÷
Í¿²¼	êîòðð	íìíòðð	ïíòíð	îëêòíè	ïîèòïç
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	êìòðð	íêìòêð	ïíòíð	îéîòëè	ïíêòîç
	êëòðð	íéëòêð	ïíòíð	îèðòèí	ïìðòìï
	êêòðð	íèêòéð	ïíòíð	îèçòïë	ïììòëè
	êéòðð	íçéòçð	ïíòíð	îçéòëë	ïìèòéè
	êèòðð	ìðçòîð	ïíòíð	íðêòðí	ïëíòðï
	êçòðð	ìîðòêð	ïíòíð	íïìòëè	ïëéòîç
	éðòðð	ìííòîð	ïíòíð	ííííòîè	ïêïòêì
	éïòðð	ììíòçð	ïíòíð	ííîòðë	ïêêòðí
	éîòðð	ìëëòéð	ïíòíð	íìðòçð	ïéðòìë
	éíòðð	ìêéòêð	ïíòíð	íìçòèí	ïéìòçï
	éìòðð	ìéçòêð	ïíòíð	íëèòèí	ïéçòìï
	éëòðð	ìçïòèð	ïíòíð	íêéòçè	ïèíòçç
	éêòðð	ëðìòðð	ïíòíð	íééòïí	ïèèòëê
	ééòðð	ëïêòìð	ïíòíð	íèêòìí	ïçíòîï
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	éçòðð	ëìïòëð	ïíòíð	ìðëòîë	îðîòêí
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	èíòðð	ëçíòîð	ïíòíð	ììíòðí	îîîòðï
	èìòðð	êðêòìð	ïíòíð	ìëíòçí	îîêòçê
	èëòðð	êïçòéð	ïíòíð	ìêíòçè	îíïòçë
	èêòðð	êííòîð	ïíòíð	ìéìòðí	îíéòðï
	èéòðð	êìêòéð	ïíòíð	ìèìòïë	îìîòðè
	èèòðð	êêðòìð	ïíòíð	ìçìòìí	îìéòîî
	èçòðð	êéìòîð	ïíòíð	ëðìòéè	îëîòìç
	çðòðð	êèèòïð	ïíòíð	ëïëòíð	îëéòêð
	çïòðð	éðîòïð	ïíòíð	ëîëòèð	îêîòçð
	çîòðð	éïêòîð	ïíòíð	ëíêòîë	îêèòïè
	çíòðð	éíðòëð	ïíòíð	ëìéòðð	îéíòëð

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	ØóÐ·´» Ü®·ª·²¹ Î»-·-¬¿²½»	
	øº¬÷	øµ·°-÷	øµ·°-÷	øµ·°-÷	ø¬±²-÷
Í¿²¼	çìòðð	éììòçð	ïíòíð	ëëéòèð	îéèòçð
	çëòðð	éëçòìð	ïíòíð	ëêèòêè	îèìòíì
	çêòðð	ééìòðð	ïíòíð	ëéçòêí	îèçòèï
	çéòðð	éèèòèð	ïíòíð	ëçðòéí	îçëòíê
	çèòðð	èðíòêð	ïíòíð	êðïòèí	íððòçï
	ççòðð	èïèòêð	ïíòíð	êïíòðè	íðêòëì
	ïððòðð	èííòéð	ïíòíð	êîììòíð	íïîòîð
	ïðïòðð	èìèòçð	ïíòíð	êíëòèð	íïéòçð
	ïðîòðð	èêìòîð	ïíòíð	êìéòîè	íîíòêì
	ïðíòðð	èéçòéð	ïíòíð	êëèòçð	íîçòìë
	ïðìòðð	èçëòîð	ïíòíð	êéðòëí	ííëòîê
	ïðëòðð	çïðòçð	ïíòíð	êèîòíð	ìïïòïë
	ïðêòðð	çîêòéð	ïíòíð	êçìòïë	íìéòðè
	ïðéòðð	çìîòêð	ïíòíð	éðêòðè	íëíòðì
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	ïðçòðð	çéìòèð	ïíòíð	éíðòîí	íêëòïï
	ïïðòðð	ççïòðð	ïíòíð	éìîòìè	íéïòîì
	ïïïòðð	ïððéòìð	ïíòíð	éëìòèè	íééòìì
	ïïîòðð	ïðîíòçð	ïíòíð	éêéòïè	íèíòëç
	ïïíòðð	ïðìðòëð	ïíòíð	ééçòëè	íèçòéè
	ïïìòðð	ïðëéòîð	ïíòíð	éçîòïí	íçêòðè
	ïïëòðð	ïðéìòïð	ïíòíð	èðìòéð	ìðîòíë

APPENDIX E
SINGLE DRILLED SHAFT EVALUATIONS

SINGLE DRILLED SHAFT EVALUATIONS

Resistance Factors for LRFD – Drilled Shafts

Axial Capacity		
Side Resistance in Clays	Alpha Method	0.45
End Bearing in Clays	Total Stress	0.40
Side Resistance in Sands	Beta Method	0.55
End Bearing in Sands	SPT Method	0.50
Uplift Resistance		
Clays	Alpha Method	0.35
Sands	Beta Method	0.45

Í¸¿º¬ ß¨·¿´ Î»-·-¬¿²½»
ÞÍ ÎÜòï Ð·»®- ïòì
íðó×²½¸ Ü·¿³»¬»® Í¸¿º¬

Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬ò
Ù¿¬»® Ì¿¾´» ¿¬ Ú´±±¼ Û´»ª¿¬·±²ã ìíçòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î$_2$ Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»-·-¬¿²½»		ϕÎ$_2$ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½»		ϕÎ$_2$ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»-·-¬¿²½»	
	ø º¬÷	øµ·°-÷	øµ·°-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷
	ðòðð	ðòðð	ðòîð	ðòîð	ðòïð	ðòïð	ðòðë	ðòðð	ðòðð
	ïòðð	ðòèð	ïïòéì	ïîòëì	êòîé	êòíï	íòïê	ðòíê	ðòïè
	îòðð	ïòéî	ïìòíð	ïêòðî	èòðï	èòïð	ìòðë	ðòéé	ðòíç
Í¿²¼	íòðð	îòèð	ïêòîê	ïçòðê	çòëí	çòêé	ìòèì	ïòîê	ðòêí
	ìòðð	íòçè	ïéòïè	îïòïê	ïðòëè	ïðòéè	ëòíç	ïòéç	ðòçð
	ëòðð	ëòíî	ïéòïè	îîòëð	ïïòîë	ïïòëî	ëòéê	îòíç	ïòîð
	êòðð	éòìè	ïéòïè	îìòêê	ïîòíí	ïðòéé	ëòíç	íòïë	ïòëè
	éòðð	çòêì	ïéòïè	îêòèî	ïíòìï	ïïòéì	ëòèé	íòçï	ïòçë
	èòðð	ïïòèð	ïéòïè	îèòçè	ïìòìç	ïîòéï	êòíê	ìòêê	îòíí
	çòðð	ïíòçê	ïéòïè	íïòïì	ïëòëé	ïíòêç	êòèì	ëòìî	îòéï
	ïðòðð	ïêòïî	ïéòïè	ííòíð	ïêòêë	ïìòêê	éòíí	êòïé	íòðç
	ïïòðð	ïèòîè	ïéòïè	íëòìê	ïéòéí	ïëòêí	éòèî	êòçí	íòìé
	ïîòðð	îðòìì	ïéòïè	íéòêî	ïèòèï	ïêòêð	èòíð	éòêç	íòèì
	ïíòðð	îîòêð	ïéòïè	íçòéè	ïçòèç	ïéòëé	èòéç	èòìì	ìòîî
	ïìòðð	îìòéê	ïéòïè	ìïòçì	îðòçé	ïèòëë	çòîé	çòîð	ìòêð
	ïëòðð	îêòçî	ïéòïè	ììòïð	îîòðë	ïçòëî	çòéê	çòçë	ìòçè
	ïêòðð	îçòðè	ïéòïè	ìêòîê	îíòïí	îðòìç	ïðòîë	ïðòéï	ëòíê
	ïéòðð	íïòîì	ïéòïè	ìèòìî	îìòîï	îïòìê	ïðòéí	ïïòìé	ëòéì
Ý´¿§	ïèòðð	ííòìð	ïéòïè	ëðòëè	îëòîç	îîòìí	ïïòîî	ïîòîî	êòïï
	ïçòðð	íëòëê	ïéòïè	ëîòéì	îêòíé	îíòìï	ïïòéð	ïîòçè	êòìç
	îðòðð	íéòéî	ïéòïè	ëìòçð	îéòìë	îìòíè	ïîòïç	ïíòéì	êòèé
	îïòðð	íçòèè	ïéòïè	ëéòðê	îèòëí	îëòíë	ïîòêè	ïìòëð	éòîë
	îîòðð	ìîòðì	ïéòïè	ëçòîî	îçòêï	îêòíî	ïíòïê	ïëòîê	éòêí
	îíòðð	ììòîð	ïéòïè	êïòíè	íðòêç	îéòîç	ïíòêë	ïêòðî	èòðð
	îìòðð	ìêòíê	ïéòïè	êíòëì	íïòéé	îèòîé	ïìòïí	ïêòéè	èòíè
	îëòðð	ìèòëî	ïéòïè	êëòéð	íîòèë	îçòîì	ïìòêî	ïéòëì	èòéê
	îêòðð	ëðòêè	ïéòïè	êéòèê	ííòçí	íðòîï	ïëòïï	ïèòîé	çòïì
	îéòðð	ëîòèì	ìïòðê	çíòçð	ìêòçë	ìðòéì	îðòíé	ïçòðí	çòëï
	îèòðð	ëëòðð	êèòçî	ïîíòçî	êïòçê	ëíòîè	îêòìï	ïçòéè	çòèç
	îçòðð	ëéòïê	çîòèð	ïìçòçê	éìòçè	êìòïé	íîòðç	îðòëì	ïðòîé
	íðòðð	ëçòíî	ïðìòêë	ïêíòçé	èïòçè	êçòéî	íìòèê	îïòîç	ïðòêë
	íïòðð	êïòìè	ïðìòêë	ïêêòïí	èíòðé	éðòðç	íëòðë	îîòðë	ïïòðí
Í¿²¼	íîòðð	êíòêì	ïðìòêë	ïêèòîç	èìòïë	éïòíï	íëòêë	îîòèî	ïïòìï
	ííòðð	êëòèð	ïðìòêë	ïéðòìë	èëòîî	éîòëð	íêòîë	îíòëé	ïïòéç
	íìòðð	êèòðé	ïðìòêë	ïéîòéî	èêòíê	éíòéì	íêòèé	îìòíé	ïîòïè
	íëòðð	éðòíè	ïðìòêë	ïéëòðí	èéòëî	éìòçê	íéòìè	îëòïé	ïîòëè
	íêòðð	éîòéê	ïðìòêë	ïééòìï	èèòéð	éêòïç	íèòïð	îëòçè	ïîòçç
	íéòðð	éëòïë	ïðìòêë	ïéçòèð	èçòçð	éèòìê	íçòîí	îêòèî	ïíòìï
	íèòðð	ééòêí	ïðìòêë	ïèîòîè	çïòïì	éçòëè	íçòéç	îéòêí	ïíòèî
	íçòðð	èðòïë	ïðìòêë	ïèìòèð	çîòìð	èðòêè	ìðòíì	îèòìç	ïìòîì

Í¸¿º¬ ß¨·¿´ Î»--·-¬¿²½»
ÞíÎÛòï Ð·»®- ïóì
íðó×²½¸ Ü·¿³»¬»® Í¸¿º¬

Ûó¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬ò
Ó¿¨·³«³ Ì¿®¹»¬ ¿¬ Ú´±±¼ Û´»ª¿¬·±²ã ìíçòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»--·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»--·-¬¿²½»		¢Î² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»--·-¬¿²½»		¢Î² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»--·-¬¿²½»	
	øº¬÷	øµ·° -÷	øµ·° -÷	øµ·° -÷	ø¬±² -÷	øµ·° -÷	ø¬±² -÷	øµ·° -÷	ø¬±² -÷
Í¿²¼	ìðòðð	ïïêòëê	ïðìòéì	îîïòíð	ïïðòêë	ïïðòèê	ëëòìí	ìêòèì	îíòìî
	ìïòðð	ïîíòìî	ïðìòéì	îîèòïê	ïïìòðè	ïïìòêì	ëéòíî	ìçòçî	îìòçê
	ìîòðð	ïíðòìì	ïðìòéì	îíëòïè	ïïéòëç	ïïèòëð	ëçòîë	ëíòðè	îêòëì
	ìíòðð	ïíéòêî	ïðìòéì	îìîòíê	ïîïòïè	ïîîòìë	êïòîî	ëêòíï	îèòïê
	ììòðð	ïììòçî	ïðìòéì	îìçòêê	ïîìòèí	ïîêòìê	êíòîí	ëçòêð	îçòèð
	ìëòðð	ïëîòíè	ïðìòéì	îëéòïî	ïîèòëê	ïíðòëê	êëòîè	êîòçê	íïòìè
	ìêòðð	ïêðòðð	ïðìòéì	îêìòéì	ïíîòíé	ïíìòéë	êéòíè	êêòíè	ííòïç
	ìéòðð	ïêéòéê	ïðìòéì	îéîòëð	ïíêòîë	ïíçòðî	êçòëï	êçòèè	íìòçì
	ìèòðð	ïéëòêè	ïðìòéì	îèðòìî	ïìðòîï	ïìíòíè	éïòêç	éíòìì	íêòéî
	ìçòðð	ïèíòéì	ïðìòéì	îèèòìè	ïììòîì	ïìéòèï	éíòçï	ééòðé	íèòëí
	ëðòðð	ïçïòçì	ïðìòéì	îçêòêè	ïìèòíì	ïëîòíí	éêòïê	èðòéê	ìðòíè
	ëïòðð	îððòíð	ïðìòéì	íðëòðì	ïëîòëî	ïëêòçî	éèòìê	èìòëî	ìîòîê
	ëîòðð	îðèòèî	ïðìòéì	íïíòëê	ïëêòéè	ïêïòêï	èðòèð	èèòíë	ììòïè
	ëíòðð	îïéòìè	ïðìòéì	íîîòîî	ïêïòïï	ïêêòíé	èíòïè	çîòîë	ìêòïí
	ëìòðð	îîêòîè	ïðìòéì	ííïòðî	ïêëòëï	ïéïòîï	èëòêð	çêòîï	ìèòïï
	ëëòðð	îíëòîî	ïðìòéì	ííçòçè	ïêççç	ïéêòïì	èèòðé	ïððòîì	ëðòïî
	ëêòðð	îììòíì	ïðìòéì	íìçòðè	ïéìòëì	ïèïòïì	çðòëé	ïðìòíì	ëîòïé
	ëéòðð	îëíòêð	ïðìòéì	íëèòíì	ïéçòïé	ïèêòîí	çíòïî	ïðèòëð	ëìòîë
	ëèòðð	îêíòðð	ïðìòéì	íêéòéì	ïèíòèé	ïçïòìð	çëòéð	ïïîòéí	ëêòíé
	ëçòðð	îéîòëê	ïðìòéì	íééòíð	ïèèòêë	ïçêòêê	çèòíí	ïïéòðì	ëèòëî
	êðòðð	îèîòîê	ïðìòéì	íèéòðð	ïçíòëð	îðîòðð	ïðïòðð	ïîïòìð	êðòéð
	êïòðð	îçîòïð	ïðìòéì	íçêòèì	ïçèòìî	îðéòìï	ïðíòéð	ïîëòèì	êîòçï
	êîòðð	íðîòïð	ïðìòéì	ìðêòèì	îðíòìî	îïîòçï	ïðêòìë	ïíðòíì	êëòïê
	êíòðð	íïîòîê	ïðìòéì	ìïéòðð	îðèòëð	îïèòëð	ïðçòîë	ïíìòçð	êéòìë
	êìòðð	íîîòëê	ïðìòéì	ìîéòíð	îïíòêë	îîìòïê	ïïîòðè	ïíçòëì	êçòéé
	êëòðð	ííííòðð	ïðìòéì	ìíéòéì	îïèòèé	îîçòçð	ïïìòçë	ïììòîí	éîòïï
	êêòðð	íììòêð	ïðìòéì	ììèòíì	îîìòïé	îíëòéí	ïïéòèé	ïìçòðð	éìòëð
	êéòðð	íëìòíê	ïðìòéì	ìëçòïð	îîçòëë	îìïòêë	ïîðòèí	ïëíòèë	éêòçî
	êèòðð	íêëòîì	ïðìòéì	ìéðòðð	îíëòðð	îìèòéì	ïîíòèî	ïêèòêî	éçòîè
	êçòðð	íéêòíð	ïðìòéì	ìèïòðð	îìðòëî	îëðòèð	ïîêòìï	ïêìòêé	èîòïê
	éðòðð	íèéòéð	ïðìòéì	ìçîòìì	îìêòîî	îëéòèè	ïîçòçì	ïéðòéê	èëòíè
	éïòðð	íççòèì	ïðìòéì	ëðìòëè	îëîòîç	îêêòïî	ïíííòðê	ïéíòèê	èêòçí
	éîòðð	ìïðòíí	ïðìòéì	ëïëòðê	îëéòëí	îéîòíí	ïíêòïî	ïéçòðí	èçòëï
	éíòðð	ìîïòçè	ïðìòéì	ëîêòéî	îêíòíê	îéèòèì	ïíçòìî	ïèìòíè	çîòïç
	éìòðð	ìííòéê	ïðìòéì	ëíèòëð	îêçòîë	îèëòìî	ïìîòéê	ïèçòëè	çìòéç
	éëòðð	ììëòéð	ïðìòéì	ëëðòìì	îéëòîî	îçïòèç	ïìëòçì	ïçìòçë	çéòìé
	éêòðð	ìëéòèð	ïðìòéì	ëêîòëì	îèïòîé	îçèòìï	ïìçòîé	îððòíç	ïððòîð
	ééòðð	ìéðòðì	ïðìòéì	ëéìòéè	îèéòíç	íðëòîè	ïëîòêì	îðëòèç	ïðîòçë
	éèòðð	ìèîòìî	ïðìòéì	ëèéòïê	îçíòëè	íïïòðç	ïëëòëì	îïïòìé	ïðëéì

Í¸¿º¬ ß¨·¿´ Î»·¬¿²½»
Þí Î Üò ï Ð·»® ïòì
íðò ¨ ²½¸ Ü·¿³»¬»® Í¸¿º¬

Û¬·³¿¬»¼ Þ¿» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬
Ê¿´«»® Ì¿¾´» ¿¬ Ú±±¬ Û´»ª¿¬·±²ã ìíçòð º¬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»·¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î₂ Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»·¬¿²½»		φÎ₂ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»·ª» Î»·¬¿²½»		φÎ₂ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»·¬¿²½»	
	øº¬÷	øµ·°÷	øµ·°÷	øµ·°÷	ø¬±²÷	øµ·°÷	ø¬±²÷	øµ·°÷	ø¬±²÷
Í¿²¼	éçòðð	ìçìòçê	ïðìòéì	ëççòéð	îççòèë	íïèòçè	ïëçòìç	îïéòïî	ïðèòëê
	èðòðð	ëðéòêê	ïðìòéì	êïîòìð	íðêòîð	íîëòçé	ïêîòçè	îîîòèí	ïïïòìî
	èïòðð	ëîðòëð	ïðìòéì	êîëòîì	íïîòêî	íííòðí	ïêêòëï	îîèòêï	ïïìòíð
	èîòðð	ëííòìè	ïðìòéì	êíèòîî	íïçòïï	íìðòïé	ïéðòðè	îííòìë	ïïéòîí
	èíòðð	ëìêòêî	ïðìòéì	êëïòíê	íîëòêè	íìéòìð	ïéíòéð	îìðòíê	ïîðòïè
	èìòðð	ëëçòçð	ïðìòéì	êêìòêì	ííîòíî	íëìòéð	ïééòíë	îìêòíì	ïîíòïé
	èëòðð	ëéíòíì	ïðìòéì	êéèòðè	ííçòðì	íêîòðç	ïèïòðë	îëîòíç	ïîêòïç
	èêòðð	ëèêòçî	ïðìòéì	êçïòêê	íìëòèí	íêçòëê	ïèìòéè	îëèòëð	ïîçòîë
	èéòðð	êððòêê	ïðìòéì	éðëòìð	íëîòéð	ííéòïî	ïèèòëê	îêìòêè	ïíîòíì
	èèòðð	êïìòëì	ïðìòéì	éïçòîè	íëçòêì	íèìòéë	ïçîòíè	îéðòçí	ïíëòìê
	èçòðð	êîèòëê	ïðìòéì	éííòíð	íêêòêë	íçîòìê	ïçêòîí	îééòîì	ïíèòêî
	çðòðð	êìîòéì	ïðìòéì	éìéòìè	íéíòéì	ìððòîê	îððòïí	îèíòêî	ïìïòèï
	çïòðð	êëéòðè	ïðìòéì	éêïòèî	íèðòçï	ìðèòïë	îðìòðé	îçðòðé	ïìëòðì
	çîòðð	êéïòëê	ïðìòéì	ééêòíð	íèèòïë	ìïêòïï	îðèòðê	îçêòëç	ïìèòîç
	çíòðð	êèêòïè	ïðìòéì	éçðòçî	íçëòìê	ìîìòïë	îïîòðè	íðíòïé	ïëïòëè
	çìòðð	éððòçê	ïðìòéì	èðëòéð	ìðîòèë	ìííòîè	îïêòïì	íðçòèî	ïëìòçï
	çëòðð	éïëòèè	ïðìòéì	èîðòêî	ìïðòíï	ììðòìç	îîðòîì	íïêòëí	ïëèòîé
	çêòðð	éíðòçê	ïðìòéì	èíëòéð	ìïéòèë	ììèòéè	îîìòíç	íîíòíî	ïêïòêê
	çéòðð	éìêòîð	ïðìòéì	èëðòçì	ìîëòìé	ìëéòïê	îîèòëè	ííðòïé	ïêëòðç
	çèòðð	éêïòëê	ïðìòéì	èêêòíð	ììíòïë	ìêëòêï	îííòèï	ííéòðç	ïêèòëì
	ççòðð	éééòðè	ïðìòéì	èèïòèî	ììðòçï	ìéìòïë	îíéòðé	íììòðé	ïéîòðì
	ïððòðð	éçîòéê	ïðìòéì	èçéòëð	ììèòéë	ìèîòéé	îììòíç	íëïòïí	ïéëòëê
	ïðïòðð	èðèòëè	ïðìòéì	çïíòíî	ìëêòêê	ìçïòìé	îìëòéì	íëèòîë	ïéçòïî
	ïðîòðð	èîìòëê	ïðìòéì	çîçòíð	ìêìòêë	ëððòîê	îëðòïí	íêëòìì	ïèîòéî
	ïðíòðð	èìðòêè	ïðìòéì	çìëòìî	ìéîòéï	ëðçòïí	îëìòëê	íéîòêç	ïèêòíë
	ïðìòðð	èëêòçì	ïðìòéì	çêîòêè	ìèðòèì	ëïèòðé	îëçòðì	íèðòðï	ïçðòðð
	ïðëòðð	èéíòíê	ïðìòéì	çéèòïð	ìèçòðë	ëîéòïð	îêíòëë	íèéòìð	ïçíòéð
	ïðêòðð	èèçòçì	ïðìòéì	ççìòêè	ìçéòíì	ëíêòîî	îêèòïï	íçìòèê	ïçéòìí
	ïðéòðð	çðêòêê	ïðìòéì	ïðïïòìð	ëðëòéð	ëìëòìî	îéîòéï	ìðîòíè	îðïòïç
	ïðèòðð	çîíòëî	ïðìòéì	ïðîèòîê	ëïìòïí	ëëìòêç	îééòíë	ìðçòçé	îðìòçè
	ïðçòðð	çìðòëì	ïðìòéì	ïðìëòîè	ëîîòêì	ëêìòðë	îèîòðí	ìïéòêí	îðèòèï
	ïïðòðð	çëéòéð	ïðìòéì	ïðêîòìì	ëíïòîî	ëéíòìç	îèêòéì	ìîëòíë	îïîòêé
	ïïïòðð	çééòðî	ïðìòéì	ïðéçòéê	ëíçòèè	ëèíòðî	îçïòëï	ìííòïì	îïêòëé
	ïïîòðð	ççîòìè	ïðìòéì	ïðçéòîî	ëìèòêï	ëçîòêî	îçêòíï	ììïòðð	îîðòëð
	ïïíòðð	ïðïðòïð	ïðìòéì	ïïïìòèì	ëëéòìî	êðîòíï	íðïòïë	ììèòçí	îîìòìê
	ïïìòðð	ïðîéòèê	ïðìòéì	ïïíîòêð	ëêêòíð	êïîòðè	íðêòðì	ìëêòçî	îîèòìê

Í¸¿º¬ ß¨·¿´ Î»-·-¬¿²½»
Þí Î Ü óï Ð·»®- ïòì
íê󷲽¸ Ü·¿³»¬»® Í¸¿º¬

Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíçòð º¬ò
É¿¬»® Ì¿¾´» ¿¬ Ú´±±¼ Û´»ª¿¬·±²ã ìíçòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î ² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»-·-¬¿²½»		ϕ Î ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½»		ϕ Î ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»-·-¬¿²½»	
	øº¬÷	øµ·°-÷	øµ·°-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷
	ðòðð	ðòðð	ðòîð	ðòîð	ðòïð	ðòïð	ðòðë	ðòðð	ðòðð
	ïòðð	ðòçê	ïéòêì	ïèòêð	çòíð	çòíë	ìòêé	ðòìí	ðòîî
	îòðð	îòðè	îïòíð	îíòíè	ïïòêç	ïïòéç	ëòçð	ðòçì	ðòìé
Í¿²¼	íòðð	íòíê	îíòêì	îéòðð	ïíòëð	ïíòêé	êòèí	ïòëï	ðòéê
	ìòðð	ìòéè	îìòéì	îçòëî	ïìòéê	ïëòðð	éòëð	îòïë	ïòðè
	ëòðð	êòíè	îìòéì	íïòïî	ïëòëê	ïëòèè	éòçì	îòèé	ïòìì
	êòðð	èòçè	îìòéì	ííòéî	ïêòèê	ïìòëè	éòîç	íòéè	ïòèç
	éòðð	ïïòëê	îìòéì	íêòíð	ïèòïë	ïëòéì	éòèé	ìòêè	îòíì
	èòðð	ïìòïê	îìòéì	íèòçð	ïçòìë	ïêòçï	èòìë	ëòëç	îòèð
	çòðð	ïêòéê	îìòéì	ìïòëð	îðòéë	ïèòðè	çòðì	êòëð	íòîë
	ïðòðð	ïçòíì	îìòéì	ììòðè	îîòðì	ïçòîì	çòêî	éòìï	íòéð
	ïïòðð	îïòçì	îìòéì	ìêòêè	îíòíì	îðòìï	ïðòîð	èòíî	ìòïê
	ïîòðð	îìòëî	îìòéì	ìçòîê	îìòêí	îïòëé	ïðòéè	çòîî	ìòêï
	ïíòðð	îéòïî	îìòéì	ëïòèê	îëòçí	îîòéì	ïïòíé	ïðòïí	ëòðé
	ïìòðð	îçòéî	îìòéì	ëìòìê	îéòîí	îíòçï	ïïòçë	ïïòðì	ëòëî
	ïëòðð	íîòíð	îìòéì	ëéòðì	îèòëî	îëòðé	ïîòëí	ïïòçì	ëòçé
	ïêòðð	íìòçð	îìòéì	ëçòêì	îçòèî	îêòîì	ïíòïî	ïîòèë	êòìí
	ïéòðð	íéòìè	îìòéì	êîòîî	íïòïï	îéòìð	ïíòéð	ïíòéê	êòèè
Ý´¿§	ïèòðð	ìðòðè	îìòéì	êìòèî	íîòìï	îèòëé	ïìòîç	ïìòêé	éòíí
	ïçòðð	ìîòêè	îìòéì	êéòìî	ííòéï	îçòéì	ïìòèé	ïëòëè	éòéç
	îðòðð	ìëòîê	îìòéì	éðòðð	íëòðð	íðòçð	ïëòìë	ïêòìè	èòîì
	îïòðð	ìéòèê	îìòéì	éîòêð	íêòíð	íîòðé	ïêòðì	ïéòíç	èòêç
	îîòðð	ëðòìì	îìòéì	éëòïè	íéòëç	ííòîí	ïêòêî	ïèòîç	çòïë
	îíòðð	ëíòðì	îìòéì	ééòéè	íèòèç	íìòìð	ïéòîð	ïçòîð	çòêð
	îìòðð	ëëòêì	îìòéì	èðòíè	ìðòïç	íëòëé	ïéòéç	îðòïï	ïðòðê
	îëòðð	ëèòîî	îìòéì	èîòçê	ìïòìè	íêòéí	ïèòíé	îïòðî	ïðòëï
	îêòðð	êðòèî	ëïòìè	ïïîòíð	ëêòïë	ìèòêð	îìòíð	îïòçí	ïðòçê
	îéòðð	êíòìð	èîòðê	ïìëòìê	éîòéí	êïòçç	íïòðð	îîòèí	ïïòìï
	îèòðð	êêòðð	ïïêòìì	ïèîòìì	çïòîî	éêòçï	íèòìê	îíòéì	ïïòèé
	îçòðð	êèòêð	ïíçòíè	îðéòçè	ïðíòçç	èéòîê	ìíòêí	îìòêë	ïîòíî
	íðòðð	éïòïè	ïëðòèì	îîîòðî	ïïïòðï	çíòðï	ìêòëð	îëòëë	ïîòéè
	íïòðð	éíòéè	ïëðòèì	îîìòêî	ïïîòíï	çìòïè	ìéòðç	îêòìê	ïíòîí
	íîòðð	èðòìð	ïëðòèì	îíïòîì	ïïëòêî	ïïîòçð	ëêòìë	îçòìì	ïìòéî
	ííòðð	èéòîî	ïëðòèì	îíèòðê	ïïçòðí	ïïêòêë	ëèòíí	íîòëï	ïêòîë
	íìòðð	çìòîð	ïëðòèì	îìëòðì	ïîîòëî	ïîðòìç	êðòîë	íëòêê	ïéòèí
Í¿²¼	íëòðð	ïðïòíê	ïëðòèì	îëîòîð	ïîêòïð	ïîìòíí	êîòïï	íèòèé	ïçòìì
	íêòðð	ïðèòéð	ïëðòèì	îëçòëì	ïîçòéé	ïîèòìé	êìòïí	ìîòïè	îïòðç
	íéòðð	ïïêòîì	ïëðòèì	îêéòðè	ïííòëì	ïíîòêï	êêòíï	ìëòëé	îîòéè
	íèòðð	ïîíòçì	ïëðòèì	îéìòéè	ïíéòíç	ïíêòèë	êèòìî	ìçòðí	îìòëî
	íçòðð	ïíïòèî	ïëðòèì	îèîòêê	ïìïòîí	ïìïòïè	éðòëç	ëîòìè	îêòîç

Í¸¿º¬ ß¨·¿´ Î»·¬¿²½»
Þí Î Üò ï Ð·»® ïòì
íêò¨²½¸ Ü·¿³»¬»® Í¸¿º¬

Ü»¬»®³·²»¼ Þ¿» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ì íçò𠺬
É¿¬»® Ì¿¾´» ¿¬ Ú±±¬·²¹ Û´»ª¿¬·±² ã ì íçò𠺬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»·¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î ₂ Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»·¬¿²½»		ϕÎ ₂ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»·ª» Î»·¬¿²½»		ϕÎ ₂ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»·¬¿²½»	
	øº¬÷	øµ·°÷	øµ·°÷	øµ·°÷	ø¬±²÷	øµ·°÷	ø¬±²÷	øµ·°÷	ø¬±²÷
Í¿²¼	ìðòðð	ïíçòèè	ïëðòèì	îçðòéî	ïìëòíê	ïìëòêï	éîòèï	ëêòîï	îèòïð
	ìïòðð	ïìèòïî	ïëðòèì	îçèòçê	ïìçòìè	ïëðòïë	éëòðé	ëçòçï	îçòçê
	ìîòðð	ïëêòëì	ïëðòèì	íðéòíè	ïëíòêç	ïëìòéè	ééòíç	êíòéð	íïòèë
	ìíòðð	ïêëòïì	ïëðòèì	íïëòçè	ïëéòçç	ïëçòëï	éçòéë	êéòëé	ííòéç
	ììòðð	ïéíòçð	ïëðòèì	íîìòéì	ïêîòíé	ïêìòíí	èîòïê	éïòëî	íëòéê
	ìëòðð	ïèîòèê	ïëðòèì	íííòéð	ïêêòèë	ïêçòîë	èìòêí	éëòëë	íéòéé
	ìêòðð	ïçîòðð	ïëðòèì	íìîòèì	ïéïòìî	ïéìòîè	èéòïì	éçòêê	íçòèí
	ìéòðð	îðïòíî	ïëðòèì	íëîòïê	ïéêòðè	ïéçòìï	èçòéð	èíòèë	ìïòçí
	ìèòðð	îïðòèð	ïëðòèì	íêïòêì	ïèðòèî	ïèìòêî	çîòíï	èèòïî	ììòðê
	ìçòðð	îîðòìè	ïëðòèì	íéïòíî	ïèëòêê	ïèçòçì	çìòçé	çîòìè	ìêòîì
	ëðòðð	îíðòíì	ïëðòèì	íèïòïè	ïçðòëç	ïçëòíé	çéòêè	çêòçï	ìèòìê
	ëïòðð	îìðòíê	ïëðòèì	íçïòîð	ïçëòêð	îððòèè	ïððòìì	ïðïòìî	ëðòéï
	ëîòðð	îëðòëè	ïëðòèì	ìðïòìî	îððòéï	îðêòëð	ïðíòîë	ïðêòðî	ëíòðï
	ëíòðð	îêðòçê	ïëðòèì	ìïïòèð	îðëòçð	îïîòîï	ïðêòïð	ïïðòêç	ëëòíë
	ëìòðð	îéïòëì	ïëðòèì	ìîîòíè	îïïòïç	îïèòðí	ïðçòðï	ïïëòìë	ëéòéí
	ëëòðð	îèîòîè	ïëðòèì	ìííòïî	îïêòêë	îîíòçí	ïïïòçé	ïîðòîç	êðòïì
	ëêòðð	îçíòîð	ïëðòèì	ìììòðì	îîîòðî	îîçòçì	ïïìòçé	ïîëòîð	êîòêð
	ëéòðð	íðìòîî	ïëðòèì	ìëëòïê	îîéòëè	îíêòðê	ïïèòðí	ïíðòîð	êëòïð
	ëèòðð	íïëòêð	ïëðòèì	ìêêòìì	îííòîî	îìîòîê	ïîïòïí	ïíëòèî	êéòêì
	ëçòðð	íîéòðê	ïëðòèì	ìééòçð	îíèòçë	îìèòëê	ïîìòîè	ïìðòìì	éðòîî
	êðòðð	ííèòéð	ïëðòèì	ìèçòëì	îììòéé	îëëòçé	ïîéòìè	ïìëòêè	éîòèì
	êïòðð	íëðòëî	ïëðòèì	ëðïòíê	îëðòêè	îêïòìé	ïíðòéí	ïëðòçç	éëòëð
	êîòðð	íêîòëî	ïëðòèì	ëïíòíê	îëêòêè	îêèòðé	ïííòðí	ïëêòíç	éèòîð
	êíòðð	íéìòéð	ïëðòèì	ëîëòëì	îêîòéé	îéìòéé	ïíêòíè	ïêïòèè	èðòçì
	êìòðð	íèéòðê	ïëðòèì	ëíéòçð	îêèòçë	îèïòëê	ïìðòéè	ïêéòìì	èíòéî
	êëòðð	íççòêð	ïëðòèì	ëëðòìì	îéëòîî	îèèòìê	ïììòîí	ïéíòðè	èêòëì
	êêòðð	ìïîòíî	ïëðòèì	ëêíòïê	îèïòëè	îçëòìê	ïìéòéì	ïéèòèð	èçòìð
	êéòðð	ìîëòîî	ïëðòèì	ëéêòðê	îèèòðí	íðîòëë	ïëïòîè	ïèìòêï	çîòíð
	êèòðð	ìíèòíð	ïëðòèì	ëèçòïì	îçìòëé	íðçòéë	ïëìòèé	ïçðòëð	çëòîë
	êçòðð	ìëïòëê	ïëðòèì	êðîòìð	íðïòîð	íïéòîð	ïëèòëî	ïçêòìê	çèòîí
	éðòðð	ìêëòçè	ïëðòèì	êïëòèî	íðéòçï	íîìòìî	ïêîòîï	îðîòëð	ïðïòîë
	éïòðð	ìéèòêð	ïëðòèì	êîçòìì	íïìòéî	ííïòçï	ïêëòçê	îðèòêí	ïðìòíî
	éîòðð	ìçîòíð	ïëðòèì	êìíòîì	íîïòêî	ííçòëð	ïêçòéë	îïìòèì	ïðéòìî
	éíòðð	ëðêòîê	ïëðòèì	êëéòîð	ííèòêè	íìéòïè	ïéíòìç	îîðòîç	ïïðòëê
	éìòðð	ëîïòìê	ïëðòèì	êéïòíê	ííëòèè	íëëòçé	ïééòëì	îîêòïç	ïïíòéë
	éëòðð	ëíëòìì	ïëðòèì	êèêòèê	íìîòìì	íêíòîì	ïèïòïî	îííòçì	ïïéòðé
	éêòðð	ëëïòíê	ïëðòèì	éððòîð	íìçòïç	íéðòìí	ïèìòëï	îíçòíé	ïîïòïí
	ééòðð	ëêèòïð	ïëðòèì	éïëòìì	íëêòïï	íééòèð	ïèèòïè	îìëòðè	ïîìòëï
	éèòðð	ëèëòçî	ïëðòèì	éíîòéê	íêîòìì	íèëòïé	ïçïòìì	îëïòëè	ïîéòðì

Í¸¿º¬ ß¨·¿´ Î»·¬¿²½»
Þí Î Üó ï Ð·»®- ïóì
íêóײ½¸ Ü·¿³»¬»® Í¸¿º¬

Û¬·³¿¬»¼ Þ¿» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ìíç𠺬
Ë¿¬»® Ì¿¾´» ¿¬ Û´±±¼ Û´»ª¿¬·±²ã ìíç𠺬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»·¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î ² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»·¬¿²½»		ϕÎ ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»·ª» Î»·¬¿²½»		ϕÎ ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»·¬¿²½»	
	øº¬÷	øµ·°÷	øµ·°÷	øµ·°÷	ø¬±²÷	øµ·°÷	ø¬±²÷	øµ·°÷	ø¬±²÷
Í¿²¼	éçòðð	ëçíòçê	ïëðòèì	éììòèð	íéîòìð	íçëòíê	ïçéòêè	îêðòëì	ïíðòîé
	èðòðð	êðçòïè	ïëðòèì	éêðòðî	íèðòðï	ìðíòéí	îðïòèê	îêéòíç	ïííòéð
	èïòðð	êîìòêð	ïëðòèì	ééëòìì	íèéòéî	ìïîòîï	îðêòïï	îéìòíí	ïíéòïé
	èîòðð	êìðòïè	ïëðòèì	éçïòðî	íçëòëï	ìîðòéè	îïðòíç	îèïòíì	ïìðòêé
	èíòðð	êëëòçì	ïëðòèì	èðêòéè	ìðíòíç	ìîçòìë	îïìòéî	îèèòìí	ïììòîî
	èìòðð	êéïòèè	ïëðòèì	èîîòéî	ìïïòíê	ìíèòîï	îïçòïï	îçëòêï	ïìéòèð
	èëòðð	êèèòðð	ïëðòèì	èíèòèì	ìïçòìî	ììéòðè	îîíòëì	íðîòèê	ïëïòìí
	èêòðð	éðìòíð	ïëðòèì	èëëòïì	ìîéòëé	ìëêòðë	îîèòðî	íïðòîð	ïëëòïð
	èéòðð	éîðòéè	ïëðòèì	èéïòêî	ìíëòèï	ìêëòïï	îíîòëë	íïéòêï	ïëèòèï
	èèòðð	éíéòìì	ïëðòèì	èèèòîè	ìììòïì	ìéìòîé	îíéòïì	íîëòïï	ïêîòëë
	èçòðð	éëìòîè	ïëðòèì	çðëòïî	ìëîòëê	ìèíòëí	îìïòéé	ííîòêç	ïêêòíì
	çðòðð	ééïòíð	ïëðòèì	çîîòïì	ìêïòðé	ìçîòçð	îìêòìë	íìðòíë	ïéðòïé
	çïòðð	éèèòëð	ïëðòèì	çíçòíì	ìêçòêé	ëðîòíê	îëïòïè	íìèòðç	ïéìòðì
	çîòðð	èðëòèê	ïëðòèì	çëêòéð	ìéèòíë	ëïïòçð	îëëòçë	íëëòçð	ïééòçë
	çíòðð	èîíòìî	ïëðòèì	çéìòîê	ìèéòïí	ëîïòëê	îêðòéè	íêíòèð	ïèïòçð
	çìòðð	èìïòïê	ïëðòèì	ççîòðð	ìçêòðð	ëíïòíî	îêëòêê	íéïòéè	ïèëòèç
	çëòðð	èëçòðê	ïëðòèì	ïððçòçð	ëðìòçë	ëìïòïê	îéðòëè	íéçòèì	ïèçòçî
	çêòðð	èééòïê	ïëðòèì	ïðîèòðð	ëïìòðð	ëëïòïî	îéëòêê	íèéòçè	ïçíòçç
	çéòðð	èçëòìî	ïëðòèì	ïðìêòîê	ëîíòïí	ëêïòïê	îèðòëè	íçêòîð	ïçèòïð
	çèòðð	çïíòèè	ïëðòèì	ïðêìòéî	ëíîòíê	ëéïòíï	îèëòêê	ìðìòëï	îðîòîë
	ççòðð	çíîòëð	ïëðòèì	ïðèíòíì	ëìïòêé	ëèïòëê	îçðòéè	ìïîòèç	îðêòìì
	ïððòðð	çëïòíî	ïëðòèì	ïïðîòïê	ëëïòðè	ëçïòçï	îçëòçë	ìîïòíë	îïðòêè
	ïðïòðð	çéðòíð	ïëðòèì	ïïîïòïì	ëêðòëé	êðîòíë	íðïòïé	ìîçòçð	îïìòçë
	ïðîòðð	çèçòìê	ïëðòèì	ïïìðòíð	ëéðòïë	êïîòèè	íðêòìì	ìíèòëî	îïçòîê
	ïðíòðð	ïððèòèî	ïëðòèì	ïïëçòêê	ëèçòèí	êîíòëí	íïïòéé	ììéòîí	îîíòêï
	ïðìòðð	ïðîèòíì	ïëðòèì	ïïéçòïè	ëèçòëç	êíìòîé	íïéòïí	ìëêòðï	îîèòðï
	ïðëòðð	ïðìèòðì	ïëðòèì	ïïçèòèè	ëççòìì	êìëòïð	íîîòëë	ìêìòèè	îíîòìì
	ïðêòðð	ïðêéòçî	ïëðòèì	ïîïèòéê	êðçòíè	êëêòðì	íîèòðî	ìéíòèî	îíêòçï
	ïðéòðð	ïðèéòçè	ïëðòèì	ïîíèòèî	êïçòìï	êêéòðé	íííòëí	ìèîòèë	îìïòìí
	ïðèòðð	ïïðèòîì	ïëðòèì	ïîëçòðè	êîçòëì	êéèòîï	ííçòïï	ìçïòçé	îìëòçè
	ïðçòðð	ïïîèòêê	ïëðòèì	ïîéçòëð	êíçòéë	êèçòìì	íììòéî	ëðïòïê	îëðòëè
	ïïðòðð	ïïìçòîì	ïëðòèì	ïíððòðè	êëðòðì	éððòéê	íëðòíè	ëïðòìî	îëëòîï
	ïïïòðð	ïïéðòðî	ïëðòèì	ïíîðòèê	êêðòìí	éïîòïç	íëêòïð	ëïçòéé	îêçòèè
	ïïîòðð	ïïçðòçè	ïëðòèì	ïíìïòèî	êéðòçï	éîíòéî	íêïòèê	ëîçòîð	îêìòêð
	ïïíòðð	ïîïîòïî	ïëðòèì	ïíêîòçê	êèïòìè	éíëòíë	íêéòêé	ëíèòéï	îêçòíê

Í¸¿º¬ ß¨·¿´ Î»·¬¿²½»
ÞÍ ÎÜòï Ð·»® ïóì
ì îóײ½¸ Ü·¿³»¬»® Í¸¿º¬

Û¬·³¿¬»¼ Þ¿» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ì í çòð º¬ò
Ù®±«²¼ Í«®º¿½» Û´»ª¿¬·±² ã ì í çòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»·¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î₂ Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»·¬¿²½»		φÎ₂ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»·ª» Î»·¬¿²½»		φÎ₂ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»·¬¿²½»	
	øº¬÷	øµ·° ÷	øµ·° ÷	øµ·° ÷	ø¬±² ÷	øµ·° ÷	ø¬±² ÷	øµ·° ÷	ø¬±² ÷
Í¿²¼	ðòðð	ðòðð	ðòíð	ðòíð	ðòïë	ðòïë	ðòðè	ðòðð	ðòðð
	ïòðð	ïòïî	îëòêè	îêòèð	ïíòìð	ïíòìê	êòéí	ðòëð	ðòîë
	îòðð	îòìî	îçòéè	íîòîð	ïêòïð	ïêòîî	èòïï	ïòðç	ðòëì
	íòðð	íòçð	íîòìî	íêòíî	ïèòïê	ïèòíê	çòïè	ïòéê	ðòèè
	ìòðð	ëòëè	ííòêè	íçòîê	ïçòêí	ïçòçï	çòçë	îòëï	ïòîê
	ëòðð	éòìì	ííòêè	ìïòïî	îðòëê	îðòçí	ïðòìé	íòíë	ïòêé
Ý´¿§	êòðð	ïðòìè	ííòêè	ììòïê	îîòðè	ïèòçí	çòìé	ìòìï	îòîï
	éòðð	ïíòëð	ííòêè	ìéòïè	îíòëç	îðòîç	ïðòïë	ëòìé	îòéí
	èòðð	ïêòëî	ííòêè	ëðòîð	îëòïð	îïòêè	ïðòèí	êòëí	íòîê
	çòðð	ïçòëì	ííòêè	ëíòîî	îêòêï	îíòðï	ïïòëð	éòëè	íòéç
	ïðòðð	îîòëê	ííòêè	ëêòîì	îèòïî	îìòíé	ïîòïè	èòêì	ìòíî
	ïïòðð	îëòêð	ííòêè	ëçòîè	îçòêì	îëòéì	ïîòèé	çòéð	ìòèë
	ïîòðð	îèòêî	ííòêè	êîòíð	íïòïë	îéòïð	ïíòëë	ïðòéê	ëòíè
	ïíòðð	íïòêì	ííòêè	êëòíî	íîòêê	îèòìë	ïìòîí	ïïòèî	ëòçï
	ïìòðð	íìòêê	ííòêè	êèòíì	íìòïé	îçòèï	ïìòçï	ïîòèè	êòìì
	ïëòðð	íéòêè	ííòêè	éïòíê	íëòêè	íïòïé	ïëòëç	ïíòçí	êòçé
	ïêòðð	ìðòéî	ííòêè	éìòìð	íéòîð	íîòëí	ïêòîé	ïëòðð	éòëð
	ïéòðð	ìíòéì	ííòêè	ééòìî	íèòéï	ííòçð	ïêòçë	ïêòðë	èòðí
	ïèòðð	ìêòéê	ííòêè	èðòìì	ìðòîî	íëòîê	ïéòêí	ïéòïï	èòëê
	ïçòðð	ìçòéè	ííòêè	èíòìê	ìïòéí	íêòêî	ïèòíï	ïèòïé	çòðè
	îðòðð	ëîòèð	ííòêè	èêòìè	ìíòîì	íéòçè	ïèòçç	ïçòîî	çòêï
	îïòðð	ëëòèì	ííòêè	èçòëî	ììòéê	íçòíì	ïçòêé	îðòîç	ïðòïì
	îîòðð	ëèòèê	ííòêè	çîòëì	ìêòîé	ìðòéð	îðòíë	îïòíë	ïðòêé
	îíòðð	êïòèè	ííòêè	çëòëê	ìéòéè	ìîòðê	îïòðí	îîòìð	ïïòîð
	îìòðð	êìòçð	ííòêè	çèòëè	ìçòîç	ìíòìî	îïòéï	îíòìê	ïïòéí
	îëòðð	êéòçî	êìòïè	ïíîòïð	êêòðë	ëêòçè	îèòìç	îìòëî	ïîòîê
	îêòðð	éðòçê	çèòëî	ïêçòìè	èìòéì	éîòðè	íêòðì	îëòëè	ïîòéç
	îéòðð	éíòçè	ïíêòêê	îïðòêì	ïðëòíî	èèòéð	ììòíë	îêòêì	ïíòíî
	îèòðð	ééòðð	ïéðòçè	îìéòçè	ïîíòçç	ïðíòéç	ëïòèç	îéòêç	ïíòèë
	îçòðð	èðòðî	ïçíòêê	îéíòèè	ïíêòçì	ïïìòíð	ëéòïë	îèòéë	ïìòíè
	íðòðð	èíòðê	îðëòíð	îèèòíê	ïììòïè	ïîðòîì	êðòïî	îçòèî	ïìòçï
	íïòðð	èêòðè	îðëòíð	îçïòíè	ïìëòêç	ïîïòêð	êðòèð	íðòèé	ïëòìì
Í¿²¼	íîòðð	çíòèð	îðëòíð	îççòïð	ïìçòëë	ïîìòêí	êïòìç	íïòíë	ïëòçé
	ííòðð	ïðïòêì	îðëòíð	íðéòðì	ïëíòëî	ïëðòéì	êëòíé	íîòéç	ïêòìç
	íìòðð	ïðçòçð	îðëòíð	íïëòîð	ïëéòêð	ïëëòïî	êéòêî	ííòïë	îðòèð
	íëòðð	ïïèòîê	îðëòíð	íîíòëê	ïêïòéè	ïëçòèí	êçòçï	íëòìë	îîòêè
	íêòðð	ïîêòèî	îðëòíð	ííîòïî	ïêêòðê	ïêìòìç	èîòîé	íçòîï	îíòêð
	íéòðð	ïíëòêð	îðëòíð	íìðòçð	ïéðòìë	ïêçòìé	èìòêè	ëíòïê	îêòèè
	íèòðð	ïììòëè	îðëòíð	íìçòèè	ïéìòçì	ïéìòíï	èéòïë	ëéòîð	îèòêð
	íçòðð	ïëíòéè	îðëòíð	íëçòðè	ïéçòëì	ïéçòíé	èçòêè	êïòìì	íðòéé

Í¸¿º¬ ß¨·¿´ Î»--¬¿²½»
Þí Î Üò ï Ð·»®- ïóì
ì îóx²½¸ Ü·¿³»¬»® Í¸¿º¬

Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ì í çò ð ºó
É¿¬»® Ì¿¾´» ¿¬ Ú´±±¼ Û´»ª¿¬·±² ã ì í çò ð ºó

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»--·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î Ì±¬¿´ Ò±³·²¿´ Ü»-·¹²¿´ ß¨·¿´ Î»--·-¬¿²½»		¢ Î Ì±¬¿´ Ú¿½¬±®»¼ Ü»-·¹²¿´ ß¨·¿´ Ý±³°®»--·ª» Î»--·-¬¿²½»		¢ Î Ì±¬¿´ Ú¿½¬±®»¼ Ü»-·¹²¿´ ß¨·¿´ Ë°´·º¬ Î»--·-¬¿²½»	
	øº¬÷	øµ·°-÷	øµ·°-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷
Í¿²¼	ìðòðð	ïêíòïè	îðëòíð	íêèòìè	ïèìòîì	ïèìòëì	çîòîé	êëòëé	íîòéè
	ìïòðð	ïéîòèð	îðëòíð	íéèòïð	ïèçòðë	ïèçòèí	çìòçï	êçòçð	íìòçë
	ìîòðð	ïèîòêî	îðëòíð	íèéòçî	ïçíòçê	ïçëòîí	çéòêï	éìòíî	íéòïê
	ìíòðð	ïçîòêê	îðëòíð	íçéòçê	ïçèòçè	îððòéë	ïððòíé	éèòèí	íçòìî
	ììòðð	îðîòçð	îðëòíð	ìðèòîð	îðìòïð	îðêòíè	ïðíòïç	èíòìì	ìïòéî
	ìëòðð	îïíòíì	îðëòíð	ìïèòêì	îðçòíî	îïîòïî	ïðêòðê	èèòïì	ììòðé
	ìêòðð	îîìòðð	îðëòíð	ìîçòíð	îïìòêë	îïéòçç	ïðèòçç	çîòçì	ìêòìé
	ìéòðð	îííòèê	îðëòíð	ììðòïê	îîðòðè	îîíòçê	ïïïòçè	çéòèî	ìèòçï
	ìèòðð	îìëòçì	îðëòíð	ìëïòîì	îîëòêî	îíðòðë	ïïëòðí	ïðîòèï	ëïòìð
	ìçòðð	îëéòîî	îðëòíð	ìêîòëî	îíïòîê	îíêòîê	ïïèòïí	ïðéòèç	ëíòçì
	ëðòðð	îêèòéî	îðëòíð	ìéìòðî	îíéòðï	îìîòëè	ïîïòîç	ïïíòðê	ëêòëí
	ëïòðð	îèðòìî	îðëòíð	ìèëòéî	îìîòèê	îìçòðî	ïîìòëï	ïïèòíí	ëçòïê
	ëîòðð	îçîòíì	îðëòíð	ìçéòêì	îìèòèî	îëëòëé	ïîéòéç	ïîíòêç	êïòèì
	ëíòðð	íðìòìê	îðëòíð	ëðçòéê	îëìòèè	îêîòîì	ïíïòïî	ïîçòïì	êìòëé
	ëìòðð	íïêòéè	îðëòíð	ëîîòðè	îêïòðì	îêçòðî	ïíìòëï	ïíìòêç	êéòíì
	ëëòðð	íîçòíî	îðëòíð	ëííòêî	îêéòíï	îéëòçï	ïíéòçê	ïìðòíí	éðòïé
	ëêòðð	íìîòðè	îðëòíð	ëìéòíè	îéíòêç	îèîòçí	ïìïòïé	ïìêòðé	éíòðì
	ëéòðð	íëëòðì	îðëòíð	ëêðòíì	îèðòïé	îçðòðê	ïìëòðí	ïëïòçð	éëòçë
	ëèòðð	íêèòîð	îðëòíð	ëéíòëð	îèêòéë	îçéòíð	ïìèòêë	ïëéòèí	éèòçï
	ëçòðð	íèïòëè	îðëòíð	ëèêòèè	îçíòìì	íðìòêê	ïëîòíí	ïêíòèë	èïòçî
	êðòðð	íçëòïê	îðëòíð	êððòìê	íððòîí	íïîòïî	ïëêòðê	ïêçòçê	èìòçè
	êïòðð	ìðèòçì	îðëòíð	êïìòîì	íðéòïî	íïçòéð	ïëçòèë	ïéêòïê	èèòðè
	êîòðð	ìîîòçî	îðëòíð	êîèòîî	íïìòïï	íîéòìï	ïêíòéï	ïèîòìé	çïòîí
	êíòðð	ìíêòçî	îðëòíð	êìîòîî	íîïòïï	ííëòîî	ïêéòêï	ïèèòèé	çìòìí
	êìòðð	ìëïòîç	îðëòíð	êëêòëè	íîèòîç	íìíòïê	ïéïòëè	ïçëòíë	çéòêé
	êëòðð	ìêêòðï	îðëòíð	êéïòíï	ííëòêë	íëïòïî	ïéëòëê	îðïòèî	ïððòçï
	êêòðð	ìèïòïê	îðëòíð	êèêòìê	ììííòîí	íëçòîê	ïéçòêí	îðèòëî	ïðìòîê
	êéòðð	ìçêòéî	îðëòíð	éðîòðî	íëïòðï	íêéòëé	ïèíòéç	îïëòíè	ïðéòêç
	êèòðð	ëïîòêè	îðëòíð	éïéòçè	íëèòçç	íéêòðê	ïèèòðí	îîîòìç	ïïïòîë
	êçòðð	ëîçòðë	îðëòíð	éíìòíë	íêéòïè	íèìòéî	ïçîòíê	îîçòèî	ïïìòçï
	éðòðð	ëìëòèí	îðëòíð	éëïòïí	íéëòëé	íçíòëë	ïçêòéè	îíéòíí	ïïèòêé
	éïòðð	ëêíòðî	îðëòíð	éêèòíî	íèìòïê	ìðîòëê	îðïòîè	îìëòðí	ïîîòëï
	éîòðð	ëèðòêî	îðëòíð	éèëòçî	íçîòçê	ìïïòéì	îðëòèé	îëîòèç	ïîêòìë
	éíòðð	ëçèòêî	îðëòíð	èðíòçî	ìðïòçê	ìîïòïð	îïðòëë	îêðòçê	ïíðòìè
	éìòðð	êïéòðí	îðëòíð	èîîòíí	ìïïòïé	ìíðòêì	îïëòíî	îêçòîï	ïíìòêï
	éëòðð	êíëòèë	îðëòíð	èìïòïë	ìîðòëè	ììðòíê	îîðòïè	îééòêë	ïíèòèí
	éêòðð	êëëòðè	îðëòíð	èêðòíè	ìíðòïç	ììëðòîê	îîëòïí	îèêòîè	ïìíòïì
	ééòðð	êéìòéî	îðëòíð	èèðòðî	ììðòðï	ìêðòíî	îíðòïê	îçëòðç	ïìéòëë
	éèòðð	êçìòéê	îðëòíð	èçðòðê	ììëðòðí	ìéðòëì	îíëòîé	íðìòðè	ïëîòðì

Í¸¿º¬ ß¨·¿´ Î»-·-¬¿²½»
Þí Î Üóï Ð·»®- ïóì
ì îó×²½¸ Ü·¿³»¬»® Í¸¿º¬

Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ì íçòð º¬ò
É¿¬»® Ì¿¾´» ¿¬ Ú´±±¼ Û´»ª¿¬·±² ã ì íçòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î ² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»-·-¬¿²½»		ϕ Î ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½»		ϕ Î ² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»-·-¬¿²½»	
	øº¬÷	øµ·° -÷	øµ·° -÷	øµ·° -÷	ø¬±² -÷	øµ·° -÷	ø¬±² -÷	øµ·° -÷	ø¬±² -÷
Í¿²¼	éçòðð	êçîòçê	îðëòíð	èçèòîê	ììçòïí	ìéëòçï	îíéòçê	íðíòçé	ïëïòçè
	èðòðð	éïðòéî	îðëòíð	çïêòðî	ìëèòðï	ìèëòêè	îìîòèì	íïïòçê	ïëëòçè
	èïòðð	éîèòêè	îðëòíð	çííòçè	ìêêòçç	ìçëòëê	îìéòéè	íîðòðì	ïêðòðî
	èîòðð	éìêòèê	îðëòíð	çëîòïê	ìéêòðè	ëðëòëê	îëîòéè	íîèòîî	ïêìòïï
	èíòðð	éêëòîê	îðëòíð	çéðòëê	ìèëòîè	ëïëòêè	îëéòèì	ííêòëð	ïêèòîë
	èìòðð	éèíòèê	îðëòíð	çèçòïê	ìçìòëè	ëîëòçï	îêîòçë	íììòèé	ïéîòìì
	èëòðð	èðîòêê	îðëòíð	ïððéòçê	ëðíòçè	ëíêòîë	îêèòïî	íëíòíí	ïéêòêé
	èêòðð	èîïòêè	îðëòíð	ïðîêòçè	ëïíòìç	ëìêòéï	îéíòíê	íêïòèç	ïèðòçë
	èéòðð	èìðòçî	îðëòíð	ïðìêòîî	ëîíòïï	ëëéòîç	îéèòêë	íéðòëë	ïèëòîè
	èèòðð	èêðòíì	îðëòíð	ïðêëòêì	ëííòèî	ëêéòçé	îèíòçç	íéçòîç	ïèçòêì
	èçòðð	èèðòðð	îðëòíð	ïðèëòíð	ëìîòêë	ëéèòéç	îèçòíç	íèèòïì	ïçìòðé
	çðòðð	èççòèì	îðëòíð	ïïðëòïì	ëëîòëé	ëèçòéð	îçìòèë	íçéòðê	ïçèòëí
	çïòðð	çïçòçð	îðëòíð	ïïîëòîð	ëêîòêð	êððòéí	íððòíé	ìðêòðç	îðíòðë
	çîòðð	çìðòïè	îðëòíð	ïïìëòìè	ëéîòéì	êïïòèç	íðëòçì	ìïëòîî	îðéòêï
	çíòðð	çêðòêê	îðëòíð	ïïêëòçê	ëèîòçè	êîíòïë	íïïòëé	ìîìòíí	îïîòîî
	çìòðð	çèïòíì	îðëòíð	ïïèêòêì	ëçíòíî	êíìòëî	íïéòîê	ììíòéì	îïêòèé
	çëòðð	ïððîòîì	îðëòíð	ïîðéòëì	êðíòéé	êìêòðî	íîíòðï	ììíòïì	îîïòëé
	çêòðð	ïðîíòíê	îðëòíð	ïîîèòêê	êïìòíí	êëéòêí	íîèòèî	ìëîòêë	îîêòíî
	çéòðð	ïðììòêê	îðëòíð	ïîìçòçê	êîìòçè	êêçòíë	ííìòêé	ìêîòîí	îíïòïî
	çèòðð	ïðêêòîð	îðëòíð	ïîéïòëð	êíëòéë	êèïòîð	íìðòêð	ìéïòçí	îíëòçê
	ççòðð	ïðèéòçî	îðëòíð	ïîçíòîî	êìêòêï	êçíòïì	íìêòëé	ìèïòéð	îìðòèë
	ïððòðð	ïïðçòèê	îðëòíð	ïíïëòïê	êëéòëè	éðëòîï	íëîòêð	ìçïòëé	îìëòéç
	ïðïòðð	ïïíîòðî	îðëòíð	ïííéòíî	êêèòêê	éïéòìð	íëèòéð	ëðïòëë	îëðòéé
	ïðîòðð	ïïëìòíè	îðëòíð	ïíëçòêè	êéçòèì	éîçòéð	íêìòèè	ëïïòêï	îëëòèð
	ïðíòðð	ïïéêòçê	îðëòíð	ïíèîòîê	êçïòïí	éìîòïï	íéïòðê	ëîïòéé	îêðòèè
	ïðìòðð	ïïççòéî	îðëòíð	ïìðëòðî	éðîòëï	éëìòêí	íééòíî	ëíîòðï	îêëòêï
	ïðëòðð	ïîîîòéî	îðëòíð	ïìîèòðî	éïìòðï	éêéòîè	íèíòêì	ëìîòíê	îéïòïè
	ïðêòðð	ïîìëòçî	îðëòíð	ïìëïòïî	éîêòëï	éèðòðì	íçðòðî	ëëîòèð	îéêòêì
	ïðéòðð	ïîêçòíí	îðëòíð	ïìéìòêî	éíéòíï	éçíòîï	íçêòìê	ëêíòíí	îèïòïé
	ïðèòðð	ïîçîòçì	îðëòíð	ïìççòèî	éìçòïî	èðëòçð	ìðîòçë	ëéíòçê	îèêòçè
	ïðçòðð	ïíïêòéê	îðëòíð	ïëîîòîê	éêïòðí	èïçòðð	ìðçòëð	ëèìòêè	îçîòíì
	ïïðòðð	ïíìðòèð	îðëòíð	ïëìêòïð	ééíòðë	èíîòíí	ìïêòïï	ëçëòëð	îçéòéë
	ïïïòðð	ïíêëòðì	îðëòíð	ïëéðòíì	éèëòïé	èìëòëê	ìîîòéè	êðêòìð	íðíòîð
	ïïîòðð	ïíèçòìè	îðëòíð	ïëçìòéè	éçéòíç	èëçòðð	ìîçòëð	êïéòìð	íðèòéð

Í¸¿º ß¨·¿´ Î»--·-¬¿²½»
Þ Í Î Üóï ó Ð·»®- ïóì
ìèó×²½¸ Ü·¿³»¬»® Í¸¿º

Û-¬· ³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ì íçòð º¬ò
Ê¿¬»® Ì¿¾´» ¿¬ Ú´±±¼ Û´»ª¿¬·±² ã ì íçòð º¬ò

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î₂ Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»-·-¬¿²½»		ϕÎ₂ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½»		ϕÎ₂ Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»-·-¬¿²½»	
	øº¬÷	øµ·°-÷	øµ·°-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷	øµ·°-÷	ø¬±²-÷
Í¿²¼	ðòðð	ðòðð	ðòìð	ðòìð	ðòîð	ðòîð	ðòïð	ðòðð	ðòðð
	ïòðð	ïòîè	íëòìè	íêòéê	ïèòíè	ïèòìì	çòîî	ðòëè	ðòîç
	îòðð	îòéê	íçòèî	ìîòëè	îïòîç	îïòìí	ïðòéï	ïòîì	ðòêî
	íòðð	ìòìê	ìîòêî	ìéòðè	îíòëì	îíòéê	ïïòèè	îòðï	ïòðð
	ìòðð	êòíè	ìíòçè	ëðòíê	îëòïè	îëòëð	ïîòéë	îòèé	ïòìì
	ëòðð	èòëî	ìíòçè	ëîòëð	îêòîë	îêòêè	ïíòíì	íòèí	ïòçî
Ý´¿§	êòðð	ïïòçê	ìíòçè	ëëòçì	îéòçé	îíòèí	ïïòçï	ëòðì	îòëî
	éòðð	ïëòìî	ìíòçè	ëçòìð	îçòéð	îëòíè	ïîòêç	êòîë	íòïî
	èòðð	ïèòèè	ìíòçè	êîòèê	íïòìí	îêòçì	ïíòìé	éòìê	íòéí
	çòðð	îîòíì	ìíòçè	êêòíî	ííòïê	îèòëð	ïìòîë	èòêé	ìòíì
	ïðòðð	îëòèð	ìíòçè	êçòéè	íìòèç	íðòðë	ïëòðí	çòèè	ìòçì
	ïïòðð	îçòîì	ìíòçè	éíòîî	íêòêï	íïòêð	ïëòèð	ïïòðç	ëòëì
	ïîòðð	íîòéð	ìíòçè	éêòêè	íèòíì	ííòïê	ïêòëè	ïîòíð	êòïë
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Í¸¿º¬ ß¨·¿´ Î»-·-¬¿²½»
ÞÍ ÎÜóï ó Ð·»®- ïóì
Ìèó·²½¸ Ü·¿³»¬»® Í¸¿º¬

Û-¬·³¿¬»¼ Þ¿-» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ì íçòð º¬
É¿¬»® Ì¿¾´» ¿¬ Ú´±±¼ Û´»ª¿¬·±² ã ì íçòð º¬

	Ü»°¬¸ Þ»´±© Ð·´» Ý¿°	Ò±³·²¿´ Í·¼» Î»-·-¬¿²½»	Ò±³·²¿´ Û²¼ Þ»¿®·²¹	Î² Ì±¬¿´ Ò±³·²¿´ Ù»±¬»½¸²·½¿´ ß¨·¿´ Î»-·-¬¿²½»		¾Î² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ý±³°®»--·ª» Î»-·-¬¿²½»		¾Î² Ì±¬¿´ Ú¿½¬±®»¼ Ù»±¬»½¸²·½¿´ ß¨·¿´ Ë°´·º¬ Î»-·-¬¿²½»	
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ìèóײ¸ Ü·¿³»¬»® Í¸¿º¬

Û¬·³¿¬»¼ Þ¿» ±º Ð·´» Ý¿° Û´»ª¿¬·±² ã ì íçòð º¬ò
É¿¬»® Ì¿¾´» ¿¬ Ú´±±¼ Û´»ª¿¬·±² ã ì íçòð º¬ò

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APPENDIX F
RECOMMENDED SOIL PARAMETERS FOR LATERAL LOAD ANALYSIS

Estimated Soil Parameters for Lateral Load Pile Analysis
Kennedy Interchanges Project – Bridge B3RD-1
Structure S0180

Ground Water Level = 444 ft.

Abutment 1

Idealized Soil Profile	Approximate Soil Elevations (ft)	Average Saturated Unit Weight (pcf)	Estimated Corrected Friction Angle (degrees)	Estimated Undrained Shear Strength (psf)	Estimated Lateral Soil Modulus Paramter, k (pci)	Estimated Soil Strain Parameter, E_{50} (%)
silty clay – fill	444-437	125	-	500	215	0.01
clay - soft to stiff	437-414.5	125	-	500	215	0.01
fine to medium sand – medium dense - dense	414.5-328	130	33.7	-	130	-

Piers 1-4

Idealized Soil Profile	Approximate Soil Elevations (ft)	Average Saturated Unit Weight (pcf)	Estimated Corrected Friction Angle (degrees)	Estimated Undrained Shear Strength (psf)	Estimated Lateral Soil Modulus Paramter, k (pci)	Estimated Soil Strain Parameter, E_{50} (%)
silty sand (fill)	444-434	125	31	-	40	-
clay - soft to stiff	434-408	125	-	500	215	0.01
fine to medium sand – medium dense to dense	408-328	130	33.7	-	130	-