Phase 1 Intermediate Design Report Hudson River PCBs Superfund Site

Attachment H – Design Analysis: Backfilling/Capping

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This attachment presents the design analysis supporting the development of backfill/cap prototypes and provides an analysis of potential backfill/cap placement equipment and techniques for the Hudson River project.

1. Development of Backfill/Cap Prototypes

Backfill prototypes consider habitat replacement/reconstruction objectives and bed stability as the basis for material selection. Cap prototypes consider many factors, including their ability to isolate PCBs, hydrodynamic influences such as water velocities and vessel forces (i.e., wakes and propeller wash), and the general influence of ice. The effect of these factors on the design analysis for backfill and cap prototypes is discussed below.

1.1 Backfill

As specified in the ROD (EPA, 2002a), backfill will consist of a nominal 12-inch layer of granular material placed on top of the residual sediment layer in dredged areas. An exception may be made if less than 12 inches of sediment are removed in a particular dredge cut, in which case, the backfill will be added to bring the final grade back to the original (pre-dredging) river bed elevation. The primary purposes of the backfill are to reduce surficial PCB concentrations and to provide substrate for habitat restoration. Thus, backfill material is primarily designed using a stable bed approach (i.e., defined for design purposes as being in geomorphic equilibrium during the 2-year flood event), with the goal of providing backfill with stability characteristics similar to the native sediments. In some cases, the backfill will provide the desired habitat characteristics, while in others, the backfill will be designed as stable material upon which natural deposition and revegetation will occur over time (see Section 3.10 of the Phase 1 IDR, Habitat Replacement and Reconstruction). Backfill will not be placed in the navigation channel or in sections of the river where the resultant deeper water (following remediation) is desired for habitat purposes. Backfill will not be placed in areas where post-dredged surficial Tri+ PCB concentrations are less than 0.25 mg/kg, unless necessary to meet specific habitat goals, as these areas would already meet the substantive requirements of the ROD (EPA, 2002a) and Hudson EPS (EPA, 2004).

This section presents a general discussion of how each type of backfill will be used to meet habitat replacement and reconstruction goals, bed stability objectives and other considerations to confirm that the selected backfill is appropriate for the river environment in which it is to be placed.

1.1.1 Habitat Considerations

In 2003 and 2004, the following habitat types were delineated and assessed in candidate Phase 1 areas:

- Unconsolidated river bottom;
- Aquatic vegetation beds;
- Fringing wetlands; and
- Shorelines.

Note that additional habitat assessment activities are being performed in 2005, with results to be incorporated in the Phase 1 FDR. The primary goal of the habitat replacement and reconstruction program is to replace the functions of the habitats of the Upper Hudson River to within the range of functions found in similar physical settings in the Upper Hudson River in light of the changes in river hydrology, bathymetry, and geomorphology that may result from dredging. In support of these goals, and where deemed necessary, backfill material types will be selected to provide an appropriate substrate, as follows:

- **Type 1 Backfill – Medium-Grained Material:** This material will consist of medium to coarse sand of approximately 0.5 to 2.0 mm D_{50} with total organic content ranging from trace up to 2.5%. The acceptable TOC range of Type 1 material may be refined during the Phase 1 Final Design. Type 1 material will be used alone or in combination with other materials (cobbles, woody debris, etc.) as a substrate for aquatic vegetation bed and riverine fringing wetland habitats. This material may also be used in unconsolidated river bottom areas to provide a substrate for benthic macroinvertebrate colonization. Type 1 material can be placed at any water depth where flow velocities are below 1.5 ft/s during a 2-year flow event.

- **Type 2 Backfill – Coarse-Grained Material (Fine Gravel):** This material will consist of fine gravel of approximately 6.0 to 12.0 mm (0.25- to 0.5-inch) D_{50}. Total organic matter content is expected to be minimal. The actual TOC of Type 2 material may be refined during the Phase 1 Final Design. This material will be used to replace or reconstruct unconsolidated river bottom or aquatic vegetation bed habitat to allow benthic invertebrate recolonization and provide fish habitat. Type 2 material can be placed at any water depth where flow velocities are above 1.5 ft/s.

1.1.2 Bed Stability Considerations

The engineering consideration for the selection of backfill is the stability of the material in the river environment. The design objective is to approximate the long-term stability of the existing sediment bed (or as termed herein, a “stable bed” approach) determined by modeling an appropriate high flow condition. The ability of non-cohesive, granular backfill to resist erosion forces is mainly dependent on its grain size. Backfill stability will increase once vegetation is established and/or heavier material is placed on top of the fill. The cohesive interactions that provide stability to very fine bed materials above that expected from size alone would increase stability, but are not considered in this analysis.

Backfill will be selected to resist scour due to river flow such that the newly established bed will remain stable under typical river conditions, a “stable bed” concept. As some degree of bed mobility is natural in river systems, the backfill will not be designed to “armor” the sediment bed with an immovable layer (i.e., it will not be designed to resist extreme erosional forces when the natural bed would be expected to likewise move).

The 2-year flow event was chosen for the backfill stability design as it is generally considered to represent geomorphically balanced, stable bed conditions (Rosgen, 1996; Federal Interagency Stream Restoration Work Group, 1998). River velocities and bed shear stress during the 2-year flow were predicted using the hydrodynamic and hydraulic models developed by the GE (QEA, 1999; Connolly et al., 2000; Ziegler et al., 2000) as presented in Attachment E of the Phase 1 IDR. Due to uncertainties in the final bathymetry, existing river conditions were used in the hydraulic model (i.e., the effect of changes in river bathymetry due to dredging and backfilling were not considered). Additional modeling is planned in support of the Phase 1 Final Design using more current and accurate bathymetric data. However, the magnitude of the changes in modeling results is not expected to be sufficient to alter the underlying basis of design related to hydrodynamic forces present in the Upper Hudson River. Modeling results predict that 2-year flow velocities have a maximum value of 4.6 ft/s over Phase 1 areas, with 85% of the area having velocities less than 3 ft/s. The median (50% value) velocity and shear stress predicted by the model for the Phase 1 dredge areas are 2.5 ft/s and 40 dynes per square centimeter (dynes/cm^2), respectively.

A variety of empirical relations have been developed to determine the size of stable bed material based upon either water velocity or shear stress (i.e., tractive force). While these methods have different underlying conditions or assumptions, they do provide a range of empirically derived values upon which to evaluate appropriate sized backfill material for geomorphic stability. The relations used in assessing potential stability for backfill material under 2-year flow conditions include:

- Julien (1995) – a linear relationship between shear stress and grain size for non-cohesive sediments;
- Lane (1955) – a graphic method based on compilation of many studies relating shear stress to transportable particle size, originally developed for sizing of canal cross-sections;
- Hjulstrom (1935, as presented in Morris and Fan, 1997) – a graphic method for relating critical velocities for erosion, transport and deposition to sediment grain size;
- USACE (1994) – a graphic method suggesting maximum permissible velocity for a range of sediment gradations, based on observed bed material for channels subject to estimated maximum velocities;
- Shields Diagram (Morris and Fan, 1997)– a relationship between shear stress and grain size for incipient sediment motion, the Shields coefficient is subject to some variations depending on the specific study, bed material variability and movement determination (Buffington and Montgomery, 1997); and
- Neill (1973) – a graphic relationship for limiting velocity in terms of grain size and water depth, which assumes bed erosion until the limiting velocity for a given non-cohesive sediment bed gradation is reached.

The stable particle sizes were computed for each of these methods using a range of velocities and shear during 2-year flow conditions. Results are presented in Table H1, below.

Table H1 – Stable Backfill Grain Size at 2-Year Flow Conditions

Relative Frequency (% less than)	Velocity (ft/s)	Shear Stress (dynes/cm^2)	Stable Sediment Size (mm)					
			Julien	Lane	Hjulstrom	USACE	Shields	Neill
16	1.5	11	0.85	0.6	1.4	<0.2	2.2	<0.3
25	1.85	22	2.8	1.8	2.8	0.3	3.8	<0.3
50	2.50	40	5.0	3.5	3.5	0.8	5.4	1
75	2.85	53	6.7	6.0	4.0	1.2	8.0	2
90	3.16	64	8.1	8.5	5.0	1.6	10	3.4
100	4.62	134	17	13	7	4.2	20	12

Based on the potential range of stable sediment size values obtained from the bed stability analysis presented in Table H1, the two backfill material types (i.e., the Type 2 backfill [fine gravel, 6 to 12 mm D_{50} (0.25- to 0.5-inch)] for high-velocity areas, and the Type 1 backfill [medium to coarse sand, 0.5 to 2 mm D_{50}] for the low-velocity areas would provide a stable bed over the range of predicted velocities during a 2-year event, as long they were used in the appropriate areas. As discussed above, it is envisioned that Type 1 backfill will be placed in areas with velocities up to 1.5 ft/s, while Type 2 backfill will be used in the rest of the areas (note that Type 2 backfill could also be used in areas with velocities less than 1.5 ft/s, if habitat considerations require so). As such, the specific type of backfill material to be placed in the dredged areas will be based on desired habitat replacement/reconstruction objectives, as described in Section 3.10 of the Phase 1 IDR. Further details regarding the observed habitat conditions in the river reaches, and the resulting rationale to use the various backfill types therein, will be presented in the Phase 1 FDR.

1.1.3 Other Considerations

Consolidation of both in-situ sediments and the backfill material is an important factor affecting overall stability. Since the residual sediment layer will have been subsurface material prior to dredging, some degree of consolidation would already have occurred. In rare instances, certain riverbed areas may contain sediments with low-bearing capacity such that the sediments will not support the backfill material. In these circumstances, alternate construction techniques such as multiple lifts, time-phased lifts, and/or geotextile base layering will be evaluated. This is not expected to occur based on data collected thus far from the SEDC Program; however, this will be reassessed in the Phase 1 Final Design using all available data. Backfill material is not expected to undergo any appreciable consolidation since no material being used for backfill construction is fine-grained material (defined as material with greater than 50% by weight passing a # 200 sieve) (Palermo et al., 1999).

1.1.4 Summary of Backfill Types

The two potential backfill types are summarized in Table H2, below.

Table H2 – Backfill Conceptual Types

Backfill Type	Habitat Objective
Type 1 - Medium-Grained Material (12-inch layer of medium to coarse sand; 0.5 to 2 mm D_{50})*	Used to promote benthic macroinvertebrate recolonization and as a substrate for aquatic vegetation beds and riverine fringing wetland habitats.
Type 2 - Coarse-Grained Material (12-inch layer of fine gravel; 6 to 12 mm D_{50})*	Used to promote benthic invertebrate recolonization and fish habitat.
No Backfill	Deep-water habitats, navigation channels.

* Refer to Section 3.10 of the Phase 1 IDR for further details on specific habitat objectives and use criteria for these materials types.

1.2 Capping

Capping, in the context of the Hudson River project, plays the role of an engineering contingency to be used in dredge areas where the Residuals Performance Standard is not met. Capping of the residual sediments will then be implemented as a means of reducing the accessibility and availability of PCBs within the system by providing a physical barrier between the residual sediment and the overlying water column. The locations where caps would be applied are described in Section 3.9 of the Phase 1 IDR.

1.2.1 Design Objectives

The design objectives for the sub-aqueous engineered caps as specified in the Hudson EPS (EPA, 2004) include:

- *"Physically isolate the residual sediments from indigenous benthos and minimize bioturbation of the residual sediments;*
- *Resist erosion due to currents, vessel wakes and waves, propeller wash, and ice rafting, etc. and stabilize the contaminated sediments (i.e., prevent resuspension and migration of the contaminated sediments);*
- *Minimize or eliminate the flux of contaminants into the water column;*
- *Maintain integrity among the individual cap layers/components (e.g., address consolidation of compressible materials);*
- *Include consideration of additional protective measures and institutional controls that are needed (e.g., additional controls for caps constructed in any area where future navigation dredging may be necessary, notifications to boaters not to drop anchor in capped areas, etc)."*

The cap design also must address the following elements:

- Selection and characterization of materials for cap construction;
- Equipment and placement techniques to be used for cap construction;
- Appropriate monitoring and management program, including construction monitoring during cap placement, followed by a long-term monitoring and maintenance program (which will include periodic inspections and actions that may be required based on the inspection results); and
- Ability to isolate the contaminated sediments chemically such that the concentration of Tri+ PCBs in the upper 6 inches of the cap is 0.25 mg/kg or less upon placement.

For purposes of design and construction of caps, the above objectives will be satisfied so long as the cap meets the basis of design set forth for the two cap types below.

Prototype designs were developed to account for a range of possible conditions in the river, including, but not limited to, residual sediment PCB concentration, water depth, and anticipated water velocities. Additional considerations may include location in the river (e.g., navigation channel, river banks), and habitat replacement and reconstruction objectives. Cap prototypes have been developed for two basic cap types:

- **Isolation Cap Type A,** to be placed in a CU where the average Tri+ PCB concentration after dredging is less than or equal to 6 mg/kg and capping is necessary; and

- **Isolation Cap Type B,** to be placed in a CU where the average Tri+ PCB concentration after dredging is greater than 6 mg/kg and GE and the EPA have determined that additional dredging is not required.

In the case of both cap types, the caps will be placed over sufficient portions of the CU such that the arithmetic average Tri+ PCB concentration of the uncapped nodes is 1 mg/kg or less and no individual uncapped node has a Tri+ PCB concentration at or above 15 mg/kg.

These prototypes will be "pre-designed" for the range of conditions expected to be encountered after dredging. The objective of developing these prototypes now is to allow construction planning and material procurement to proceed and allow placement after dredging without delay. The prototype designs will need to consider practical limitations and efficiency of the dredging and capping operations and account for factors such as bottom conditions, hydraulic conditions, residuals PCB concentrations, habitat replacement and reconstruction needs, and cap placement success (in completed CUs). The decision regarding appropriate cap type to be installed will be made in the field by GE's field representative (in consultation with the design engineer and subject to approval by EPA's field representative), since the actual performance of the dredge equipment and subsequent residuals concentrations will not be known until project implementation.

Basis of Design for Isolation Cap Type A

The basis of design for Isolation Cap Type A will be as follows:

- The design objectives will be achieved by installation of an armoring layer designed to withstand a minimum 10-year recurrence interval flow event.
- A filter layer (i.e., layer of material with smaller particle size to separate residuals from the armor) will be installed below (or mixed in with) the armor layer, if necessary, to prevent transport of residual sediment up through the armor material. An Isolation Cap Type A will have a total thickness of at least 12 inches when installed, which will satisfy the objective of isolating the residual sediments from indigenous benthos and limiting bioturbation of residual sediment.

Basis of Design for Isolation Cap Type B

The basis of design for Isolation Cap Type B will be as follows:

- The design objectives will be achieved by installation of an engineered isolation layer and an armor layer.
- The isolation layer will consist of material to physically and chemically isolate PCBs in contaminated sediment from the overlying water column. This layer may include a filter layer if deemed necessary for armoring purposes. An Isolation Cap Type B that includes an isolation layer that is at least 6 inches thick and have a minimum TOC content of 0.5% when installed shall satisfy the objective of reducing the flux of Tri+ PCBs from contaminated sediment into the water column. In addition, an Isolation Cap Type B will have a total thickness of at least 12 inches when installed, which will satisfy the objective of isolating the contaminated sediments from indigenous benthos and limiting bioturbation of residual sediment.
- An armoring layer will be designed to withstand a 100-year recurrence interval flow event. The armoring layer will also be designed to withstand ice events, vessel wake, and propeller wash in areas likely to be subject to such events.

1.2.2 Other Design Considerations

Similar to backfill, consolidation of cap material and underlying sediments is an important consideration affecting stability. However, considering the fact that the new sediment layer would most likely have been subsurface material prior to dredging, some degree of consolidation would have already occurred. In rare instances, certain riverbed areas may contain sediments with low-bearing capacity such that the sediments will not support the backfill material. In these circumstances, alternate construction techniques such as multiple lifts, time-phased lifts, and/or geotextile base layering will be evaluated. This is not expected to occur based on data collected thus far from the SEDC Program; however, this will be reassessed in the Phase 1 Final Design using all available data. No consolidation of cap material itself has been assumed since material being used for cap construction is not fine-grained material (defined as material with greater than 50% by weight pass a #200 sieve) (Palermo et al., 1999).

The effects of benthic organisms on the cap have been considered in the cap design. "In the biologically active sediment layer, which can extend down to depths of approximately 5 to 10 cm, rates of diffusion and particle mixing are greatly enhanced by bioturbation" (NRC, 2001). Bioturbation is a broadly defined term for the movement or alteration of sediment particles or porewater as a result of the activities of benthic organisms. It can include specific processes such as bioadvection, biodiffusion, and bioirrigation (Clarke et al., 2001). The depth to which burrowing can occur is dependent on characteristics of both the species and the substrate. More than 90% of the 240 observations for bioturbation depths in both marine and freshwater settings reported by Thoms et al. (1995) were 15 cm or less, with 80% being 10 cm or less. Even in marine settings where the organisms tend to be larger and burrow deeper, the region of sediment heavily affected by benthic organisms tends to be 5 to 15 cm, and "occasionally deeper excisions by organisms generally do not significantly affect the overall mass of contaminants or the exposure of animals living in overlying water" (NRC, 2001). It has also been observed that a sandy substrate inhibits the depth of bioturbation compared to a silty bed (Morton, 1989).

In preparation of the *Guidance for In-situ Subaqueous Capping of Contaminated Sediments* (Palermo, 1998), a survey was made of noted aquatic biologists from several research facilities around the Great Lakes. Most of the researchers found that bioturbation in a sand cap would be limited to the top 5 to 10 cm. In addition, the presence of armor stone should inhibit colonization by deeper burrowing benthic organisms (Palermo et al., 1998). Benthos at such a capped site is likely to be limited to the fine-grained, organic-rich sediments that may deposit on top of the cap or settle in the interstices of armor stone (Palermo et al., 1998).

For the prototype Hudson River caps, a bioturbation allowance of up to 6 inches (15 cm) is within acceptable limits. This is more than the expected depth for bioturbation activity in the upper Hudson River. As long as benthos cannot burrow to the depth of the residual sediment, the cap will perform as designed. For the low-velocity Type A caps, there is a net target cap thickness of 12 inches. For the medium- to high-velocity Type A caps, there is 6 inches of armor with an additional 6-inch fine gravel layer below. For the Type B caps, bioturbation will be limited to the area above the isolation material. There is 6 inches of armor layer for the low-velocity cap; and, for the medium- and high-velocity caps, a 6-inch armor layer and a 3-inch filter layer is provided for, between the residual sediment and the upper cap surface.

1.2.3 Design Evaluations

The following sections provide the basis for the selection of the three main components of the prototype caps – armor layer, filter layer, and isolation or base layer.

1.2.3.1 Armor Layer

Proper armoring is important because many of the sediment-water processes that are responsible for PCBs entering the water column or interacting with biota are only present at or near the sediment bed surface and are directly eliminated by the physical barrier of an intact cap. A properly armored cap also prevents the most obvious transport mechanism for particle bound hydrophobic contaminants such as PCBs - resuspension of particles by scour during high flow conditions. In addition to resuspension, PCBs are subjected to other sediment-water processes within the sediment bed that can contribute to vertical migration. These include molecular diffusion, ground-water induced advection, sediment mixing by river flow-induced turbulence, bioturbation, biodiffusion/bioirrigation, direct desorption, bed structure pressure-induced flows, ebullition of diagenic gases, and emergence and uprooting of aquatic plants or other processes (DePinto, 2003). With an intact cap providing physical separation between the residual sediment and overlying water, for current Hudson River conditions as much as 90% of the non-capped flux can be eliminated even without the consideration of a sorptive isolation layer (Thibodeaux, 2003).

High-river flows can cause hydraulic scour in river areas. The 10-year flow (for Isolation Cap Type A) and 100-year flow (for Isolation Cap Type B) were used as the design criteria. Unlike the stable bed used for sizing potential backfill material, for the Isolation caps, the cap armor layer is determined by a more rigorous standard of specifying material in dis-equilibrium with existing river conditions, sized for essentially no movement or loss. In addition, if Isolation Cap Type B is to be placed in locations where ice scour or vessel wake/propeller wash is likely to be dominant, the cap has been designed with consideration of such forces.

<u>Hydraulic Modeling of River Flows</u>

River velocities and bed shear stress for the 10- and 100-year flow conditions were predicted using the hydrodynamic and hydraulic models developed by GE (QEA, 1999; Connolly et al., 2000; Ziegler et al., 2000) as presented in Attachment E in the Phase 1 IDR. Existing river hydraulic conditions were used in the armor layer design without consideration of the effect of changes in river bathymetry due to dredging. Figures H1 through H8 show the velocity distributions over the Phase 1 dredge areas. These figures will be used as a basis in the determination of armor types that will be used within the dredge area following dredging (if a cap is required).

<u>Isolation Cap Type A Armor Design</u>

For the Isolation Cap Type A, the design objectives will be achieved by installation of an upper surface layer that is designed to withstand a 10-year recurrence interval flow event (e.g., 34,500 cubic feet per second [cfs] at

Fort Edward). The selection of cap armor material types also considered commonly available material types in the project area.

Because of the higher residual sediment concentrations being capped by the Isolation Caps, a more conservative armoring relation, rather than the previously discussed stable bed relation (for backfills), will be used in designing the surface material for these caps. The Isbash equation (Maynord, 1995) was used to determine the median grain size (D_{50}) for the Isolation Cap Type A armor stone as follows:

$$D_{50} = \frac{C * V^2}{\left(2g \frac{\gamma_s - \gamma_w}{\gamma_w}\right)}$$

where:

D_{50} = median stone diameter (ft)

C = Isbash constant for embedded stone (0.69)

g = acceleration due to gravity (32.2 ft/sec^2)

γ_s = specific weight of stone (165 lb/ft^3)

γ_w = specific weight of water (62.4 lb/ft^3)

V = velocity (fps)

The Isbash equation yields an armor that is essentially "non-moving" under the action of the design forces considered. For the designated velocity ranges, the resulting stone sizes based on the Isbash equation are given below in Table H3.

Table H3 - Isolation Cap Type A Armor Sizing

Velocity (ft/s)	D50 (inches)
1.5	0.18
3.5	0.96
5	1.96

Based on the range in stable stone size presented above, the Isolation Cap Type A for the low-velocity areas (<1.5 ft/s during a 10-year event) will consist of a 12-inch layer of fine gravel (6 to 12 mm, or 0.25- to 0.5-inch

D_{50}). Approximately 10% of the Phase I area has 10-year velocity less that 1.5 ft/s. The fine gravel will provide both physical isolation and armoring for the low-velocity Type A cap.

For medium- to high-velocity Isolation Cap Type A placed in areas with flow greater than 1.5 ft/s at a 10-year flow event, a 6-inch thick armor layer consisting of coarse gravel material (2-inch D_{50}) will be provided to resist the higher velocities. Note that the upper surface layer of coarse gravel should be stable to a velocity slightly exceeding 5 ft/s (which is the higher end of the maximum predicted 10-year velocity range), based on Table H3. This layer will be placed above a 6-inch base layer consisting of fine gravel, resulting in a total layer thickness of 12 inches for this cap.

Isolation Cap Type B Armor Design

For the Isolation Cap Type B, the armor layer is designed to withstand a 100-year recurrence interval flow event (e.g., 47,300 cfs at Fort Edward). The low-velocity armor for the Isolation Cap Type B is designed to withstand a maximum water velocity of 1.5 ft/s. Approximately 60% of the Phase 1 area has predicted 100-year velocities above 3.5 ft/s, falling into the high-velocity designation. The high-velocity armor for the Isolation Cap Type B is designed to withstand a water velocity of 6 ft/s. Less than 2% of the Phase 1 area has predicted velocities exceeding 6 ft/s.

Like Isolation Cap Type A, the Isbash equation (Maynord, 1995) was used to determine the median grain size (D_{50}) for the Isolation Cap armor stone. In addition, due to the higher residual concentrations that this cap is intended to protect against, an additional safety factor of 1.33 was used to account for potentially variable localized velocities within the modeled hydrodynamic grid and to account for other data uncertainties. For the designated velocity ranges, the resulting stone sizes based on the Isbash equation with and without the factor of safety are given below in Table H4, below

Table H4 - Isolation Cap Type B Armor Sizing

Armor Stone Sizing from Isbash Equation		
Velocity Range (ft/s)	D50 (inches)	D50 * FS[1] (inches)
1.5	0.18	0.23
3.5	0.96	1.27
6	2.83	3.77

Note: FS = Factor of safety = 1.33

In the low-velocity areas (<1.5 ft/s during a 100-year event), the Isolation Cap Type B armor will consist of a 6-inch layer of fine gravel (6 to 12 mm, or 0.25- to 0.5-inch D_{50}).

For the Isolation Cap Type B in medium-velocity areas (>1.5 ft/s but <3.5 ft/s), the armor layer consists of a 6-inch layer of coarse gravel (2 inches D_{50}).

For the Isolation Cap Type B in high-velocity areas (>3.5 ft/s), the armor layer consists of a 6-inch layer of cobble-sized material (4-inch D_{50}). Note that the layer thickness above accounts for guidance specifying the need for thickness to be greater or equal to 1.5 times the D_{50}, and the need to consider constructability aspects when specifying layer thickness (Maynord, 1995; Palermo et al., 1998).

Vessel Effects

Vessel effects that could potentially act on the cap include prop wash and vessel wake. Data from the NYSCC regarding vessels using the river were used to estimate vessel effects. Only those vessels with dimensions that would allow them to use the Champlain Canal (depth = 12 feet) were considered. Vessel specifications for the three classifications of watercraft used in this evaluation are presented in Table H5, below.

Table H5 - Vessel Specifications Summary Table

Watercraft	Velocity (mph)	Draft (ft)	Width (ft)	Length (ft)	Applied HP	Propeller Diameter (ft)	Depth of Propeller Axle (ft)
Tugboat	6	6	13	60	800	3	3
High-Speed Pleasure Boat	12	1.5	8	25	50	1.4	2
Cabin Cruiser	10.5	4	14	50	100	2	2

Note:
1. mph = miles per hour

Propeller Wash

Propeller wash was estimated using the equation given by Blaauw and Van de Kaa (1978), as follows:

$$V_{bp} = C_1 * U_o * \frac{D_p}{H_p}$$

where:

V_{bp} = maximum bottom velocity, expressed in ft/s

C_1 = 0.22 for non-ducted propellers

D_p = propeller diameter, expressed in feet

H_p = the distance from the propeller shaft to the channel bottom, expressed in feet

U_o = jet velocity exiting the propeller, expressed in ft/s

U_o is estimated as follows:

$$U_o = C_2 * \left(\frac{P_d}{D_p{}^2} \right)^{1/3}$$

where:

C_2 = 9.72 for non-ducted propellers

P_d = applied engine power, expressed in horsepower (hp)

In an analysis of prop wash effects at aquatic disposal sites, Clausner and Truitt (1987) noted that for practical purposes, the stable particle size computed by the above methods is probably too large. Following were some of the reasons cited by Clausner and Truitt (1987):

- It is unlikely that the craft will be operating at a high throttle within a confined channel;
- The bottom velocities are reduced if the craft is underway; and
- The length of time a craft is over a given location is probably small.

Propeller wash-induced bottom velocities were nevertheless evaluated for each vessel type at a range of water depths using the above methods. This analysis assumed that the tugboat activity would be restricted to in or near the navigation channel, and that the high-speed pleasure craft and cabin cruisers would navigate both deep and shallow waters (i.e., as shallow as 5 feet). Propeller wash from the tugboat was evaluated for water depths ranging from 10 to 30 feet. Propeller wash from the high-speed pleasure craft and cabin cruiser was evaluated for water depths ranging from 5 to 30 feet. The resulting bottom velocities are shown in Table H6, below.

Table H6 - Propeller Wash Bottom Velocities

Watercraft	Jet Velocity (ft/s)	Water Depth (ft)						
		30	25	20	15	10	7.5	5
		Bottom Velocity (ft/s)						
Tugboat	43.3	1.1	1.3	1.7	2.3	4.1	NA	NA
High-Speed Pleasure Craft	28.6	0.3	0.4	0.5	0.7	1.1	1.6	2.9
Cabin Cruiser	28.4	0.5	0.5	0.7	1.0	1.6	2.2	4.2

These velocities are within the low-high flow velocity range already anticipated for the 10- and 100-year storms being used as criteria for the Type A and Type B Isolation Caps, respectively. In general, watercraft prop wash impacts are likely to produce velocities less disruptive than flood events in high-velocity areas. In rare occurrences where a craft passes through very shallow water, some disruption to the cap surface may take place; however, there would likely be no substantial loss of cap material (due to the short impact time, the relatively small area affected by the propeller wash and the relatively high settling velocity of these materials). For example, the fine gravel used in the low-velocity Type B cap has settling velocities on the order of 1 ft/s.

Vessel Wakes

Vessel wakes can erode shorelines and shallow water areas. Vessel wakes are generated due to the pressure gradient that develops along the vessel hull as it moves through the water. The height of the wake wave depends on the vessel speed, bow and stern geometry, and the clearance between the vessel hull and channel bottom and sides. The period and direction of the wake waves is dependent on vessel speed and water depth. Vessel-generated waves decrease with increasing distance from the vessel (*Coastal Engineering Manual)* (USACE, 2002).

The USACE's *Coastal Engineering Manual* recommends reviewing published vessel wave measurement data to compare with the vessel, vessel speed, and channel conditions that most closely approach the design conditions and select a conservative wave height from these data. Two sources were reviewed for this evaluation: Maynord (2001) published vessel wake wave measurement data for vessels 16 to 20 feet in length and motors ranging from 35 to 50 hp. Wake waves were measured on Johnson Lake and on the Kenai River, Alaska. The maximum wake wave produced for all conditions was 1.07 feet.

The USACE's *Interim Report for the Upper Mississippi River – Illinois Waterway System Navigation Study, Prediction of Vessel-Generated Waves with Reference to Vessels Common to the Upper Mississippi River System* (Sorenson, 1997) summarizes vessel wake data collected from cruiser-type vessels at velocities ranging from 2 to 14 mph in water depths ranging from 10 to 33 feet. Wave height data were collected at distances ranging from 40 to 800 feet from the vessel. The maximum reported wave wake is approximately 2.1 feet.

Since only two published data sets were found that could be applied to the project environment, vessel-generated wave wake was also modeled. Several methods for modeling vessel wake are available in the literature; however, most are only applicable for deepwater conditions and non-breaking waves. Deepwater conditions are defined as Froude number (F_r) less than 0.7. Froude number is calculated by:

$$F_r = \frac{V}{\sqrt{gd}}$$

where:
V = vessel velocity, expressed in ft/s
g = gravitational constant, 32.2 ft/s^2
d = water depth at the vessel, expressed in ft

Non-breaking wave conditions exist when H_{max}/d is less than 0.6 and where H_{max} is the maximum height of the vessel-generated wave. The Froude number under the site conditions generally did not meet the criteria for deepwater conditions. Therefore, the equation given by Hochstein in USACE, 1980 was used to estimate the maximum wave height (H_{max}) generated by watercraft. The equation is given by:

$$H_{\max} = 0.0448V^2\left(\frac{D}{L}\right)^{0.5}\left(1-\frac{bD}{Ac}\right)^{-2.5}$$

where:
L = length of vessel, expressed in feet
V = vessel velocity, expressed in ft/s
Ac = cross sectional area of the channel, expressed in ft^2
D = draft of vessel, expressed in feet
b = width of vessel, expressed in feet

The wake was estimated for cross-sectional areas ranging from 2,000 to 120,000 ft^2 based on estimated channel bathymetry. The high-speed pleasure craft was found to generate the largest vessel wake, with a maximum wave height of 2.98 feet at a 2,000 ft^2 channel cross-section.

The effect of a 3-foot wave on the river bed is generation of a bottom velocity of 3.3 ft/s (calculated as the maximum horizontal velocity at edge of the bottom boundary layer, U_m). This is developed from the relationship given by Sheng and Lick, (1979):

$$U_m = \frac{\pi H}{T * \sinh\left(2\pi h / L\right)}$$

where:

H = wave height in ft

T = wave period in seconds

L = wave length in ft, and

h = water depth in ft

The resulting velocity from this equation was also confirmed using USACE's computer program, ACES (Automated Coastal Engineering System). This velocity is within the medium to high flow-velocity range already anticipated for the 10- and 100-year storms being used as criteria for the Type A and Type B isolation caps respectively. Thus, vessel wake effects are likely to produce velocities less disruptive than flood events in high-velocity areas. In rare occurrences where a craft passes through very shallow water, some disruption to the cap surface may take place; however, there would likely be no substantial loss of the cap material (due to the short impact time, the relatively small area affected by the event and the relatively high settling velocity of these materials).

Ice Effects

Ice can damage armor layers through the plucking of stones by rising and falling water levels and by ice shoving, as well as indirectly damaging armor layers through turbulent, high-flow produced beneath ice jamming. An analysis of potential ice impacts in the Phase I area was performed in June 2005 by Dr. George Ashton (see Attachment I to the Phase 1 IDR).

Findings by Dr. Ashton include:

- Dams at and upstream of Hudson Falls retain ice in those upstream reaches. This fact, combined with no observed tree scarring other than at the water line caused by floating debris, leads to a conclusion that there is no historical evidence of ice jamming from the upstream end of Rogers Island to the Thompson Island Dam;

- Frazil ice (ice in very small crystals formed in supercooled flow, slightly below 0ºC) formation upstream of Rogers Island is possible, and if transported downstream and accumulated on the underside of the ice cover could increase local velocities to a limiting critical velocity of approximately 2.0 to 2.3 fps. Above this velocity, the frazil ice would stay in the water column and be transported further downstream;

- Anchor ice (frazil ice which is distributed through the depth of the flow and attaches itself to the bottom sediments) formation is limited to areas above Rogers Island with little potential for anchor ice formation in designated dredge areas; and

- There is some potential for freezing of shallow waters (less than 2-foot depth) that could entrain sediments. However, areas of the river conductive to the thickest ice formation are also protected areas outside of the main flow where the ice may melt in place.

This limiting critical velocity for frazil ice is within the medium to high flow-velocity range already anticipated for the 10- and 100-year storms being used as criteria for the Type A and Type B Isolation Caps, respectively. Thus, normal ice effects are likely to produce velocities less disruptive than flood events in high-velocity areas. In situations where ice impacts very shallow water, some disruption to the cap surface may take place, however, it is expected that there would likely be minimal loss of sediment from the cap due to the relatively small areas affected by the event.

Further details of the ice analysis, as presented in Dr. Ashton's report, are included in Attachment I to the Phase 1 IDR.

1.2.3.2 Filter Layer

The objective of a filter layer is to protect the residual sediments or base layer from hydraulic "winnowing" (i.e., loss of smaller particles, such as the isolation material, through the void spaces in much larger material, such as the armor) and/or scour, and help to distribute the load induced by the armor layer so that the geotechnical instability of the residual sediments and/or engineered isolation layer is minimized. The filter layer is needed only for the armoring specified in the medium-velocity and high-velocity Isolation Cap Type B. The filter layer criteria, however, is used in evaluating the base layer of the medium to high-velocity Isolation Cap Type A. The Terzaghi-Vicksburg criteria (1943) are often used as guidelines in the design of a filter layer. Three criteria must be met to provide hydraulic stability. These criteria must be met when comparing the armor and filter layers, as well as the filter layer and the underlying material. These criteria are:

Armor layer $_{(A)}$ and filter layer $_{(F)}$:

$D_{15(A)} < 5D_{85(F)}$;
$20D_{15(F)} > D_{15(A)} > 5D_{15(F)}$; and
$D_{50(A)} < 25D_{50(F)}$.

Filter layer $_{(F)}$ and isolation or base layer $_{(IC)}$:

$D_{15(F)} < 5D_{85(IC)}$;
$20D_{15(IC)} > D_{15(F)} > 5D_{15(IC)}$; and
$D_{50(F)} < 25D_{50(IC)}$.

For application to Hudson River capping, among the various criteria, the relation $D_{15(A)} < 5^* D_{85(F)}$, which deals with winnowing of material through the armor, is the most important, since this controls the long-term physical stability of the cap's isolation layer.

Based on estimates of the relative size of the potential armor and underlying material, gradation for the suggested filter layer is 0.25- to 0.5-inch (6 to 12 mm) fine gravel.

According to the USACE's *Hydraulic Design of Flood Control Channels* (USACE, 1991/1994), at minimum, the filter layer thickness should be 25% of the armor stone layer thickness. In this case, assuming the 6-inch stone armor material, the resulting thickness is 1.5 inches. However, because it may not be practical to place such a thin layer underwater, a filter layer with a 3-inch thickness is used in the cap design.

1.2.3.3 Base/Isolation Layer

The type of isolation cap (i.e., Isolation Cap Type A or Type B) will be used to determine the type of base isolation layer required to control PCB migration into the water column. Since the physical presence of the cap will isolate the residual sediments from direct interaction with the water column, the possibility of scour is reduced and potential PCB mobility is primarily associated with the dissolved phase.

The Isolation Cap Type A will not include a base isolation layer specifically designed to provide a chemical barrier. Instead, this cap will provide isolation by reducing the potential for erosion of residual sediment and provide a physical barrier to direct intrusion by benthic biota. To provide a barrier to benthic intrusion, a 6-inch layer of fine gravel type material should provide adequate protection. For the low-velocity Type A caps, the base layer will be 12 inches thick; while for the medium to high-velocity Type A caps, the base layer will be a 6-inch thick fine gravel material, with another 6 inches of coarse gravel armor placed over it.

An adsorptive isolation layer is included in Isolation Cap Type B. The adsorptive layer will contain a minimum of 6 inches of fine sand with a TOC content of 0.5%. The adsorptive isolation layer is intended to control diffusive and advective flux of dissolved PCBs through the cap. Diffusive flux is a relatively slow process in which a solute moves from areas of higher concentration to areas of lower concentration due to random molecular motion, whereas for advective flux, solute mass transport is driven by fluid movement (e.g., groundwater flow). As described above in Sections 1.3.1 and 1.3.2 of this attachment, bioturbation should be limited to the armoring and filter layers above the isolation material.

Mathematical models were used to estimate flux through the isolation layer due to diffusive and advective processes, and are presented in subsequent sections.

The two major design parameters for the isolation layer of the Isolation Cap Type B are the organic carbon content (or other measure of sorptive capacity) and the layer thickness. As noted above, the minimum thickness for the isolation layer is 6 inches with a TOC content of 0.5%. The organic carbon content of the isolation layer material increases the ability of the layer to sorb PCBs, providing greater retardation of PCB migration through the layer. The thickness of the isolation layer controls the time for migration through the layer – migration time increases proportionately with thickness.

Chemical Migration by Diffusion

Diffusive flux is mathematically modeled by Fick’s law taking the general form:

$$\frac{\partial C}{\partial t} = D\left(\frac{\partial^2 C}{\partial x^2} + \frac{\partial^2 C}{\partial y^2} + \frac{\partial^2 C}{\partial z^2}\right)$$

where:

C = solute concentration

t = time

x, y, and z = the three directions of the Cartesian coordinate system

D = the diffusion coefficient

The solution for one-directional chemical flux via diffusion through the chemical isolation layer of a cap is estimated by Wang et al. (1991), Thoma et al. (1993), and Murray et al. (1994), as:

$$\frac{F_t}{F_{ss}} = 1 + 2\sum_{N=1}^{\infty} -1^N \exp\left(\frac{-D_t N^2 \rho^2 t_b}{L^2}\right)$$

where:

F_t = Flux at time ‘t’, expressed in kg/yr/m^2

F_{ss} = Steady-state flux expressed in kg/yr/m^2

t_b = breakthrough time expressed in seconds

L = cap thickness expressed in cm

D_t = transient transport effective diffusion coefficient expressed in cm/s, where $D_t = \dfrac{D_e}{R}$

R = retardation factor (unitless), where $R = \varepsilon + \rho_b K_p$

D_e = effective diffusivity expressed in cm^2/s, where $D_e = D_w e^{4/3}$

D_w = chemical diffusivity expressed in cm^2/s

ε = sediment porosity (unitless)

K_P = partitioning coefficient (L/kg), where $K_p = K_{oc} f_{oc}$

K_{oc} = chemical distribution coefficient (L/kg)

f_{oc} = fraction of organic carbon (unitless)

ρ_b = bulk density expressed in g/cm^3

Using the conservative assumption of an infinite supply of PCB in the residual sediment to calculate breakthrough (5% of maximum flux) time (t_b in seconds),and steady state (95% of maximum flux) time (t_{ss}), the above equation reduces to:

$$t_b = \frac{0.54L^2}{D_t \pi^2}$$

and

$$t_{ss} = \frac{3.69L^2}{D_t \pi^2}$$

Chemical Migration by Advection

When groundwater movement through the sediment and isolation layer occurs, solute transport occurs via both advective and diffusive processes. The following one-dimensional advective/dispersive equation incorporating a retardation factor for adsorption of PCBs was used to estimate the breakthrough and steady-state times associated with advective transport of PCBs through the isolation layer:

$$\frac{\partial C}{\partial t} = \frac{D_H}{R}\frac{\partial^2 C}{\partial x^2} - \frac{V}{R}\frac{\partial C}{\partial x}$$

The solution to this equation is given by (Bedient et al., 1985; Fetter, 1993):

$$C(x,t) = \frac{C_0}{2}\left[erfc\left(\frac{Rx - Vt}{2\sqrt{RD_H t}}\right) + \exp\left(\frac{V_x}{D_H}\right) erfc\left(\frac{Rx + Vt}{2\sqrt{RD_H t}}\right)\right]$$

where:

C = porewater concentration at location x at time t

C_o = porewater concentration in the residual layer

erfc is the complementary error function

The second term of the equation can be neglected where advective processes are the predominant mechanism of transport without introduction of measurable error (Ogata and Banks, 1961). When x is set to the isolation layer thickness (L), the equation then reduces to:

$$\frac{C}{C_o} = \frac{1}{2} erfc\left(\frac{RL - Vt}{2\sqrt{RD_H t}}\right)$$

In the presence of dissolved organic carbon, which may facilitate the transport of PCBs, a lower limit of the retardation coefficient associated with DOC-facilitated transport can be estimated as (Magee et al., 1991):

$$R = 1 + \frac{\left(K_p \rho_b / \varepsilon\right)}{\left(1 + K_{DOC} M_{DOC}\right)}$$

where:

K_{DOC} = chemical – DOC distribution coefficient (L/kg)

M_{DOC} = concentration of porewater DOC (kg/L)

Cap Design Assumptions

For the purposes of modeling mass transport through the cap during the design, the base isolation layer of the cap was assumed to have the following characteristics (see Table H7, below):

Table H7 - Isolation Cap Type B Parameter Summary Table

Parameter	Value
Thickness of isolation layer	6 inches
TOC of isolation material	0.5%
Cap Bulk Density	1.5 g/cm^3
Cap Porosity	0.4 (unitless)

Where measurable upward seepage of groundwater is present, mass transport will be dominated by advection through a cap. This assumption was tested using the dimensionless Peclet number, which indicates the relative magnitude of diffusion to advection in the cap (Palermo et al., 1998). The Peclet number for conditions in the Upper Hudson River is much greater than 1 indicating advection dominates; therefore, only the results from the advective transport analytical modeling are presented. To evaluate the performance of the cap with the above characteristics, representative conditions for the river were selected as inputs to the advective mass transport equation (see Table H8, below):

Table H8 - Sediment Parameter Summary Table for Cap Modeling

Parameter	Basis	Source
Seepage Velocity	0.18 $L/m^2/hr$	TIP Report (QEA, 1998)
Dissolved Organic Carbon	33.7 mg/L	TIP Report (QEA, 1998) Butcher and Garvey, 2004
Hydrodynamic Dispersion Coefficient	1E-10 m^2/s	Tchobanoglous and Schroeder, 1985
K_{OC}	$10^{5.4}$ L/kg	TIP Report (QEA, 1998)
K_{doc}	$10^{4.4}$ L/kg	(Koc *0.1) (OBG, 1991) (Koc *0.033) (Butcher and Garvey, 2004)
Sediment TOC	2.5%	Range 1-4% (SSAP) 1.8% (Butcher and Garvey, 2004)
TOC of Isolation Material	0.5%	To provide sufficient retardation properties; expected to be available from local sources.

These values are reasonably conservative for the Upper Hudson River. The seepage velocity used represents the highest measured rates from six locations along the Thompson Island Pool during late spring. The average for all measurements was 0.04 $L/m^2/hr$. Connolly et al. (2000) indicated a log K_{oc} of 6.26 near Fort Edward, decreasing to 5.6 at Thompson Island Pool for Tri+ PCBs. Erickson et al. (2005) estimated a Tri+ PCB log K_{oc} of 5.55. Butcher and Garvey (2004) found log K_{oc} of 5.7 or greater for all studied tri+ congeners, and the corresponding K_{doc} values ranging from 0.09 to 0.01 times K_{oc} (averaging 0.033 K_{oc}). The low K_{doc} values are consistent with the observation that paired measurements of water and sediment indicated DOC-facilitated transport was not a significant factor in determining PCB phase distribution in Hudson River water and sediments (Connolly et al., 2000).

Based upon the parameters listed above, the breakthrough time for the 6-inch isolation layer would be approximately 80 years with steady state flux achieved in 120 years. These values are specific to the conditions listed above, but do provide an estimate of the anticipated performance. Other conservative assumptions included no depletion of the residual sediment PCB source by losses either from degradation or migration, no desorption of PCB fraction within the sediment matrix and no additional retardation provided by the filter and/or armor layer material or deposition of new sediment. The continuation of sediment deposition for at least the next several decades is predicted by the sediment transport model, even if the river experiences extreme events such as the 100-year flood (Connolly et al., 2000). Based on Cesium-137 (Cs-137) profiles, ongoing sediment deposition is averaging approximately 0.5 to 1 cm/year, which, given the higher natural TOC in native sediments compared to cap material, is the equivalent of approximately 10 to 30 years of sorption capacity.

Even if breakthrough eventually occurs, the cap can provide significant reduction in flux compared to the uncapped condition. Connolly et al. (2000) had previously noted that vertical advection is not significant in the Upper Hudson River sediment bed. Thibodeaux (2003) graphically illustrated the theoretical behavior of PCB flux from sediments related to differences in mass transfer coefficients associated with various bed processes. In terms of chemodynamics, the role of capping is to decrease the mass transfer coefficient present at a site. While the decrease is certainly greater prior to breakthrough, the cap at steady state still can significantly reduce the flux of PCBs. In a comparison of generic order of magnitude flux rates, Thibodeaux (2003) found that the various processes at the surface sediment can be 2 to 3 orders of magnitude greater than the advective processes.

To compare the relative advective flux at steady state with the flux under the "without cap condition", the advective flux can be compared to a corresponding surface flux based on observed sediment-water exchange coefficients. Due to the complexities in understanding the contributions of individual processes associated with sediment water exchange of chemicals, Thibodeaux et al. (2001) proposed use of a mechanistic model of sediment-water exchange using "field-observed" exchange coefficients. Several attempts have been made to define a PCB sediment water exchange coefficient (K_f) for the Upper Hudson River for non-scour flow conditions. Connolly et al. (2000) found the winter value of K_f for Tri+ PCBs to be approximately 3 cm/day, increasing to 10 to 14 cm/day in the spring. Erickson et al. (2005) computed average K_f for 12 congeners, ranging from 2.6 to 18.8 cm/day, with an estimated Tri+ PCB exchange coefficient of 12.8 cm/day (individual determinations ranging from 1.04 to 64.6). This was in good agreement with earlier EPA (2000) estimates that averaged 12.15 cm/day (ranging from 1.96 to 44.7). Butcher and Garvey (2004) present a value of 14.8 cm/day for Tri+ PCBs in the Thompson Island Pool sediments. Note that the observed K_f value is more than 2 orders of magnitude larger than the effective transfer based solely on the molecular diffusion coefficient for PCBs in porous sediments, indicating that, even at the surface, molecular diffusion by itself is not a significant mechanism of PCB migration (Erickson et al., 2005).

The sediment-water exchange coefficient values are multiplied by the concentration difference between PCBs in the sediment pore water and the overlying water column, or if the overlying water concentration is assumed to be zero thereby maximizing the gradient, the relation simplifies to K_f times the pore water PCB concentration to derive a flux. Similarly, at steady state, the flux due to advective transport is the seepage velocity (specific discharge) times pore water PCB concentration. The 0.18 L/m^2/hr (representing the value during the maximum recorded period) corresponds to a velocity 0.43 cm/day. Using a conservative K_f of 5 cm/day and an assumed residual Tri+ PCB concentration of 10 mg/kg, if uncapped, the flux is estimated at 53 mg/m^2/yr. If capped, the steady state flux (estimated to occur after more than 100 years) for a seepage velocity of 0.43 cm/day is less than 5 mg/m^2/yr.

Even using a relatively high value for advective transport, the physical segregation provided by the cap accounts for more than 90% of the flux reduction during non-scour periods (as well as elimination of resuspension) compared to rates if the residual were in contact with the water column, with additional flux suppression being provided for nearly a century by the isolation layer. These results are in general agreement with the order of magnitude assessment of bed transport processes developed by Thibodeaux (2003).

The values presented above are for illustrative purposes only to provide a representative example of anticipated cap performance in the Upper Hudson River for the prototype Isolation Cap Type B. The values given are neither being presented, nor should they be interpreted, as numeric goals for cap performance.

1.2.3.4 Cap Selection - Other Considerations

The above-described cap types will need to be modified to include additional engineering considerations under three conditions: a) when the cap is placed along a shoreline; b) when a cap is placed within the navigational channel; and c) when a cap interfaces laterally with either the native sediment or backfill material. These conditions are discussed below.

Shoreline areas: As discussed in the steps for developing dredge prisms (Section 3.3.3.1 of the Phase 1 IDR), a 2-foot vertical cut will be made at the shoreline for dredge areas that come in contact with the shoreline. Then, the slope from the bottom of this cut to the DoC line will be 3 horizontal to 1 vertical (3:1). Following completion of dredging activities, one of the backfill or other appropriate material types will be used up to the shoreline areas and any structures placed in the river (such as sheet piling) will be removed. In shoreline areas that require capping, the cap will be constructed so that the elevation and slope of the final cap surface results in stable shoreline conditions (3:1 side-slopes). Additional protection will likely be needed for maintained shorelines disturbed by site activities (e.g., shoreline areas with existing riprap). In these areas, the shoreline will be restored to at least its pre-remediation level of protection. For natural shorelines, there are habitat treatments that may be employed, as described in Section 3.10 of the Phase 1 IDR. The shoreline stabilization design details will be completed during Phase 1 Final Design. A conceptual detail for shoreline stabilization is presented on Contract Drawing H-0052.

Navigation channel: In conformance with the Residuals Performance Standard, should a cap be placed in the navigation channel, the cap must be placed so the final surface is no higher than the prescribed channel depth and includes an indicator layer of coarse material to signal the proximity of the cap during future navigation dredging. In certain cases, additional dredging, beyond inventory and residual dredging, may be needed to deepen the area to the required depth. In these cases, the residuals will be tested to determine whether capping is still required following completion of the deepening dredging passes. These special cases will be evaluated during Phase I Final Design.

Interface with native sediment or backfill: At the lateral extent of each cap, the cap will either interface with shoreline (discussed above), native sediment in the river or areas to be backfilled. To the extent practicable, efforts will be made so that the cap smoothly transitions to the native bed or post-backfill elevation. The cap material will either taper upward or downward (depending on the relative position in comparison to other material) at a slope to be determined during the Phase 1 Final Design. To provide protection against undercutting of the cap, the upper layer of cap material (and the filter material, if used) will be placed along the slope with native material. For the interface with the backfill material, the cap will be placed first and tapered (at a slope to be determined during the Phase 1 Final Design). The backfill will then be placed covering part or all of the tapered portion of the cap. Further details of these concepts will be presented in the Phase 1 FDR.

1.2.3.5 Summary of Prototype Caps

Based on the information presented above, six prototype cap designs have been developed to address the range of conditions expected to be encountered in dredged areas. These six caps represent a combination of two PCB concentration ranges as described in Section 3.9 of the Phase 1 IDR (i.e., Isolation Cap Type A in CUs with average Tri+ PCBs $\leq$ 6 mg/kg and Isolation Cap Type B in CUs with average Tri+ PCBs > 6 mg/kg) with three flow-velocity ranges (0 to 1.5 ft/s; 1.5 to 3.5 ft/s; and greater than 3.5 ft/s), as summarized in Table H9, below.

Table H9 - Summary of Design for Prototype Caps

Cap Type	Velocity Area	Cap Materials	Thickness
Isolation Cap Type A	Low-velocity	• Fine gravel, 0.25- to 0.5-inch D_{50}.	12 inches
	Medium- to High-velocity	• Isolation layer of fine gravel, 0.25- to 0.5-inch D_{50} ; and	6 inches
		• Armor layer of coarse gravel, 2-inch D_{50}.	6 inches
Isolation Cap Type B	Low-velocity	• Isolation layer of fine sand with a TOC of approximately 0.5%; and	6 inches
		• Armor layer of fine gravel, 0.25- to 0.5-inch D_{50}.	6 inches
	Medium-velocity	• Isolation layer of fine sand with a TOC of approximately 0.5%;	6 inches
		• Filter layer of fine gravel, 0.25- to 0.5-inch D_{50}; and	3 inches
		• Armor layer of coarse gravel, 2-inch D_{50}.	6 inches
	High-velocity	• Isolation layer of fine sand with a TOC of approximately 0.5%;	6 inches
		• Filter layer of fine gravel, 0.25- to 0.5-inch D_{50}; and	3 inches
		• Armor layer of cobbles, 4-inch D_{50}.	6 inches

Note that these specifications may be refined during Final Design, based on additional data.

2. Review of Potential Placement Techniques and Equipment

The accuracy and efficiency of material placement during backfilling/capping operations are critical to promoting the effectiveness of remedial activities. During Phase 1 Intermediate Design, several backfilling/capping techniques were evaluated based on their applicability to sediment types found in the river, anticipated environmental conditions that would occur in the Upper Hudson River, and estimated accuracy in the field. To compare potentially applicable methods, a literature review and review of completed projects were conducted to supplement prior experience. Due to the different types of backfilling/capping materials and the various river regimes, several techniques are potentially applicable at the Hudson River.

This attachment provides an overview of the backfilling/capping techniques evaluated, including the following:

- Conventional equipment;
- Spreading via barge movement;
- Pipeline with baffle plate or sand box;
- Clamshell bucket;
- Submerged diffuser;
- Sand spreader barge; and
- Trémie pipes.

While surficial (i.e., at the water surface) placement techniques such as surface discharge and barge spreading are commonly used in deep water applications, and techniques such as clamshell are applicable to almost any situations, concerns over water column effects have driven some projects to use submerged discharge. Herbich (2000) notes that "If the placement of contaminated sediment by surface discharge would result in unacceptable water column effects, or if the anticipated degree of spreading and water column dispersion for either the contaminated or capping material would be unacceptable, submerged discharge is a potential control measure." Submerged discharge provides additional control and accuracy during placement and, as a result, will reduce the amount of capping material required (USAEWES, 1991). Several equipment alternatives are available for submerged discharge including subsurface placement using clamshell, submerged diffuser, sand spreader barge, and Trémie pipe.

When comparing various backfilling/capping techniques, factors to consider include navigational and positioning equipment control and the compatibility of the equipment and capping material (USAEWES, 1991). Additional equipment, such as mooring barges, electronic positioning devices, and real-time helmsman's aids, can enhance the effectiveness of the backfilling/capping activities. An example of such a system is the WINOPS system – a software program designed to aid in positioning dredges and barges during marine operation.

The various placement techniques are described below.

2.1 Surface Discharge with Conventional Equipment

Cap placement by surface discharge involves the release of capping or backfill material at the water surface allowing material to settle through the water column. The successful placement of capping material using this method depends on a number of factors, including the physical characteristics of the material being placed and site characteristics. Materials released in this manner tend to "…descend rapidly to the bottom as a dense jet with minimal short-term losses to the overlying water column" (USAEWES, 1991).

Although there are several types of surficial discharge methods, barge and pipeline placement were evaluated for the Hudson River project. Barge placement typically results in a tighter mound and less water column dispersion than pipeline placement. However, the surface discharge method does allow some of the material to become entrained in the water column during descent, which will ultimately reduce the amount of material placed in the desired area (EPA, 2002b).

Surface discharge with conventional equipment would not be appropriate for the Hudson River project due to the lack of accuracy of placement, control of equipment, and unfavorable site conditions (e.g., shallow water depths).

2.2 Spreading via Barge Movement

This method is similar to surface discharge, but controls placement of the material by slowly moving the barge during discharge and distributing the material over a specified area. Most commonly, this method involves controlling the opening of a conventional split hull barge, which results in a sprinkling action of the material. Tugs are used to move the barge slowly during release and the sediment is spread as a thin layer over a large area.

Barge movement techniques have been successfully used for the placement of predominantly coarse grained, sandy capping materials at sites in Puget Sound (Sumeri, 1989). Another location where spreading by barge was used for *in situ* capping operations was the Eagle Harbor East in Washington (Sumeri, 1995). This cap was placed to 1 to 3 feet in thickness with 275,000 cy of sand material (Palermo, 2002).

This method is not suitable for fine- to medium-grained material since such material can exit the barge relatively quickly while the barge is only partially opened. Using barges for spreading cap materials may not be suitable in shallow water depths because of the water depths needed for barge draft and door openings (EPA, 2002b). Thus, spreading via barge movement would not be appropriate to the Hudson River project

2.3 Pipeline with Baffle Plate or Sand Box

This method involves the discharge of material through a surface pipeline with the aid of a spreading device (such as baffle plate) attached to the end of the pipeline. The baffle plate or momentum plate serves two functions for discharging material during cap placement. First, the baffle plate allows for radial discharge as the material strikes the plate while exiting the pipe. Second, the angle of the plate can be adjusted to be able to maneuver the end of the pipeline in an arcing motion further controlling the placement of material (USAEWES, 1991). This method is best suited for spreading thin layers over a large area. This technique is similar to the sand box, where the device acts as a diffuser using the baffles and side boards to dissipate energy (Palermo et al., 2000). A site where this method was successfully used was the Simpson-Tacoma Kraft site in Puget Sound (Sumeri, 1989).

Hydraulic placement is well-suited to placement of thin layers over large surface areas (Palermo et al., 2000). For the Hudson River, the acreage of each cap is dependent upon residual concentration (which may or may not be a large surface area). Further, this method does not allow for the placement of the armor layers (2- to 4-inch stones) and therefore is not applicable to the entire range of Hudson River capping operations.

2.4 Clamshell Bucket

A clamshell bucket operated from a barge is a time-tested placement technology for marine operations, including cap placement. This method can be either surface or subsurface discharge as shown in the Grasse River demonstration study. The Grasse River study showed that the placement of dry, bulky capping material via clamshell was more effective and cost-efficient in achieving environmental objectives than the Trémie method of placing slurried capping material. The combination of a sophisticated clamshell positioning system (GPS/WINOPS) and experienced crane operator was found to be important to the success of cap placement (Alcoa, 2002).

Clamshell placement of cap material is also being used for the capping at the Thea Foss Wheeler Osgood Waterway project, Commencement Bay Superfund site, Tacoma, Washington. In this project, the clamshell is lowered 3 to 5 feet below water surface and slowly swung while being opened. This double action facilitates relatively even distribution of the cap material.

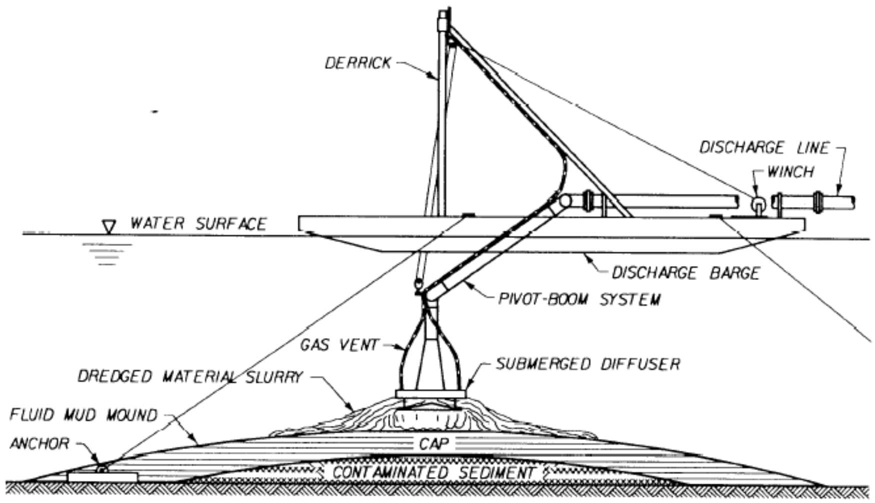
A modified clamshell bucket was also used to place cap material at the Pacific Sound Resources Superfund Site Marine Sediments Operable Unit in Seattle, Washington. Capping was completed in water depths ranging from about 0 to -35 feet MLLW, with total cap thickness varying from 4 to 7 feet. Caps were placed on relatively flat areas as well as on sloped areas of about 2:1. Capping was performed by lowering the clamshell to within 3 feet of the sediment surface, opening the clamshell, and releasing cap material. Five types of cap material were placed with the clamshell, including gravel mix, habitat mix, sand, gravel, and riprap.

This method is applicable to the Hudson River capping operations due to both the accuracy of the placement of materials and the range of materials and conditions under which the system can operate. Clamshell placement is also expected to meet the Noise Performance Standard with little to no modification to the equipment.

2.5 Submerged Diffuser

This technology was developed under the direction of the USACE Dredged Material Research Program. A submerged diffuser can be used to provide additional control for submerged pipeline discharge (Herbich, 2000). The diffuser is mounted to the end of a pipeline discharge and isolates the discharge from the majority of the water column. This method is best suited to material that is in slurry form and is illustrated on Figure H9, below. A variation of this diffuser design was used in a demonstration study at Calumet Harbor, Illinois, where it was noted that it "…significantly reduced pipeline exit velocity, confined the discharged material to the lower portion of the water column and reduced suspended solids in the upper portion of the water column" (Palermo et al., 2000). Submerged diffusers produce less turbidity than other methods that involve placement at the water surface. Submerged diffusers can place material more quickly than clamshell placement.

Figure H9 -- Submerged Diffuser System (Herbich, 2000)



This technology was also used in the Simpson Capping Project in Tacoma, Washington, which was aimed at isolating the chemical contamination present in the marine sediments and restoring the intertidal and shallow water habitat (RETEC, 2002). The capping material was placed using a diffuser and the final thickness ranged from 8.2 to 21.3 feet. Riprap was used to prevent erosion from wave action in the intertidal areas (RETEC, 2002). Results of monitoring indicate that the cap is functioning as intended.

For the Hudson River, submerged diffusers could only be used to place the finer grained backfill or cap material. Another placement method would be needed for the coarser-grained material. Submerged diffusers could accurately place cap material and document placement locations. Submerged diffusers may be less effective at placing cap material on slopes when compared to a clamshell. Finally, backfill and cap materials are likely to arrive at the site dry (i.e., with low moisture content). The addition of water would be required to use submerged diffusers. Therefore, this method is unlikely to be used solely for site operations due to the inability to place armor material.

2.6 Sand Spreader Barge

A sand spreader barge is a specialized barge used for the hydraulic spreading of sand that employs a combination of a hydraulic dredge with a submerged discharge. This process involves transporting the material by barge to the spreader; water is added to the sand, which is then pumped as slurry through a submerged pipeline. The spreader can be moved by using a winch system to cap a large area. However, this method is for sand only and may not be used for the armor material.

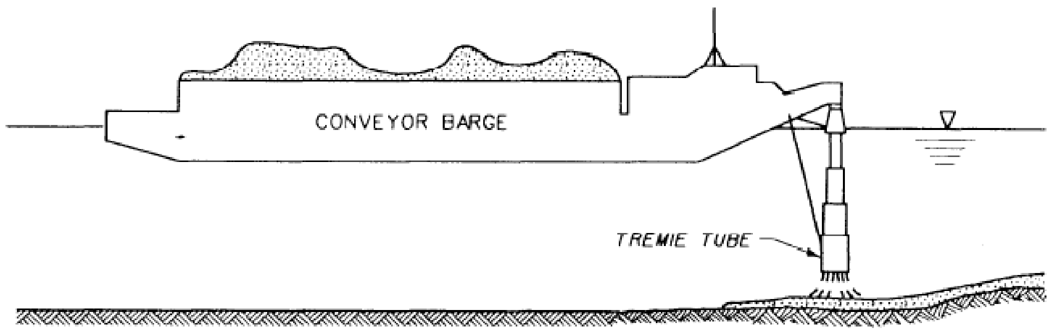
Through studies performed for the Fox River, the spreader mechanism can readily place a sand or silt-sand mixture of backfill at rates approaching 250 cy/hour and thereby cover more than an acre per day of backfill material at a thickness of 12 inches (EPA, 2004). Barge spreading has successfully been used for capping operations at Eagle Harbor, Washington (EPA, 2002b).

For the Hudson River, sand spreaders could only be used to place the finer grained backfill or cap material. Another placement method would be needed for the coarser grained material. Sand spreaders could more accurately place cap material. Sand spreaders may be less effective at placing cap material on slopes when compared to a clamshell. Sand spreaders can place material more quickly than clamshell placement, and produce equivalent or less turbidity. This method is unlikely to be used solely for site operations due to the inability to place armor material.

2.7 Trémie Pipe

The Trémie consists of a large-diameter conduit extending from the surface to just above the bottom (see Figure H10, below). This equipment improves placement accuracy and isolates the material from the water column. However, the velocity at which the material encounters the bottom is not reduced within the conduit due to the equipment typically having a large-diameter straight vertical section. The equipment, due to its size and construction, will be subject to currents and vessel wakes. Past studies have indicated that this technique results in a more controlled placement (Georgia Tech Research Corporation, 2005).

Figure H10 -- Conveyor Unloading Barge with Trémie (USAEWES, 1991)



A variation of this system was used at a capping demonstration project in Hamilton Harbor in Burlington Ontario. The Hamilton Harbor capping project consisted of a 1.64-foot thick sand cap placed over 2.47 acres using an array of Trémie tubes for sand spreading. Sand was applied in three lifts to achieve a final thickness of approximately 1.14 feet (Azcue et al., 1998). To maintain accuracy of placement of material, a system of anchors and cables was used. Sand, piled on a flat-deck barge, was placed into a hopper barge using a small front-end loader. Inside the hopper, the sand was slurried and routed into 6-inch diameter, PVC plastic tubes. The tubes extended 30 feet down, where the sand exited about 5 to 10 feet above the sediment. An anchor and winch system was used to position the barge (EPA, 2002b).

Trémie pipe placement could likely be used for the range of grain sizes planned for backfill and cap material. This method would most likely be appropriate for all components of the backfill/capping operations. Trémie pipes could accurately place cap material and document placement locations. Trémie pipes may be less effective at placing cap material on slopes as a result of the velocity at which the material will impact the sediment surface. Trémie pipes likely produce more turbidity than other subsurface placement methods because of the velocity at which cap material will impact the sediment surface. Additional information is needed on effectiveness of the technique over a range of material sizes.

2.8 Summary of Potentially Applicable Placement Techniques

The backfilling/capping technique selected for the Hudson River should be appropriate over the range of materials being installed and the conditions in which capping operations will be performed. To conduct efficient operations for both backfilling and capping, the same placement method should also be appropriate for the backfill placement operations as well. Based on a review of the various options, the clamshell method (surface or subsurface placement) is the most likely backfill/cap placement method to be used for the Hudson River. This is due to the fact that while both clamshell and Tremie pipes seem attractive based on their unique features, a clamshell is more proven in placing varying material types in conditions similar to the Hudson River. The type of clamshell, operation of the clamshell, and construction tolerances, will be evaluated further during Final Design.

2.9 Backfill and Cap Placement Plan

A backfilling/capping placement plan was developed in order to determine barge traffic requirements and volumetric production quantities during backfilling/capping operations (see Tables 3-46 and 3-47). This plan is similar to the Phase 1 Inventory and Residual Dredging Plans, which are detailed in Section 3.3 of the Phase 1 IDR. The backfill placement plan creates a schedule detailing daily placement volumes according to each dredging subarea. Placement rates are taken into consideration according to the location of each gridcell, varying cycle times and uptime according to differences in shoreline, obstructions and accessibility. These placement volumes were then used to determine the required number of barges for delivery of backfill and capping materials for each day of placement.

The placement volume for the intermediate design backfilling/capping placement plan is based on the following assumptions:

- Phase 1 total dredge area is 80 acres.
- Since backfill will not be placed in the navigational channel, the effective maximum area for backfill is 63 acres.
- For the purpose of backfill placement and barging plans, a total mass (or volume) of 217,000 tons (167,000 cy) was assumed based on the following:
    - Backfill will be placed over 40 acres
        - Assumed thickness of backfill is 12 inches.
        - A 15% contingency over the 40 acres has been assumed for engineering purposes.
        - This results in a total backfill volume of 96,000 tons (74,000 cy).
    - Capping materials will be placed over 40 acres
        - Assumed cap thickness is 15 inches.
        - A 15% contingency over the 40 acres has been assumed for engineering purposes.
        - This results in a total cap volume of 121,000 tons (93,000 cy).

- An additional 15% backfill allowance (26,000 tons or 20,000 cy) over the entire 80 acres of dredge areas will be allocated for creation of SAV beds; however, for the purposes of the backfill/cap placement plan, this additional volume (26,000 tons or 20,000 cy) is already accounted for through the overall assumption that all 80 acres in Phase 1 will receive backfill or cap and the 15% engineering purpose contingency (29,000 tons or 22,000 cy).

To determine the rates of placement, several assumptions were made concerning the abilities of the placement equipment. A 4-cy clamshell bucket with a cycle time of approximately 120 seconds was used to establish the maximum daily rate of placement. For loading, the bucket was assumed to be 90% full (on average), and during placement, a 20% bucket placement overlap (with nearby grid) was assumed. Rates of production included an uptime of 70% (which includes allowance for time lost due to barge movement, weather delays, and minor repairs). Major repairs or other operations requiring longer spans of time are assumed to be constrained to off production days - assumed to be Sundays for these schedules. Using these assumptions, a placement rate of 75 cy/hr is computed which equates to about 2,340 tons/day (1,800 cy/day).

Conditions within specific grid cells which will slow placement operations, include work within 30 feet of the shoreline which is assumed to reduce the placement rate to 60% of full operational ability (to 45 cy/hr), and obstructions which are assumed to reduce the placement rate to 50% of maximum (to 37.5 cy/hr). Regions of difficult access have a further reduction due to barge movement constraints. In order to account for the fact that the backfilling/capping operation has to always follow the completion of the dredging program at each CU, an additional reduction factor of 25% to 50% was applied on top of the previously stated efficiency factors. Backfill operations are assumed to begin approximately 3 weeks following the start of residuals dredging. This allows time for post-residuals dredging hydrographic survey, lab testing of samples and the completion of CU certification checklist.

As with the dredging plans, barge access issues within the upper regions of NTIP requires the usage of two different barge sizes. A 900-ton deck barge (i.e., a barge that can hold 900 tons of material) is assumed for the subareas EGIA, NTIP02B, NTIPO2G, and most of NTIP02F. A smaller, 500-ton barge is assumed for NTIP01, NTIP02A, NTIP02C, NTIP02D, NTIP02E, and a portion of NTIP02F.

The results from the analysis (see Tables 3-46 and 3-47) indicate that two or more barge loads are needed for over 99% of the placement operations, with four or more barge loads being needed for approximately 50% of the time, and six or more barge loads being needed for approximately 7% of the time. To decrease the number of lockages required for backfilling/capping barges, two smaller barge loads could be placed in the lock at the same time and be pushed by one tug just as long as the total length of the barges and tugs does not exceed the maximum canal length.

Note that this plan results in an upper end conservative analysis and should be considered as a planning tool only for Phase 1 IDR purposes. This plan may be revised during the Phase 1 Final Design and through contractor submittals during project implementation.

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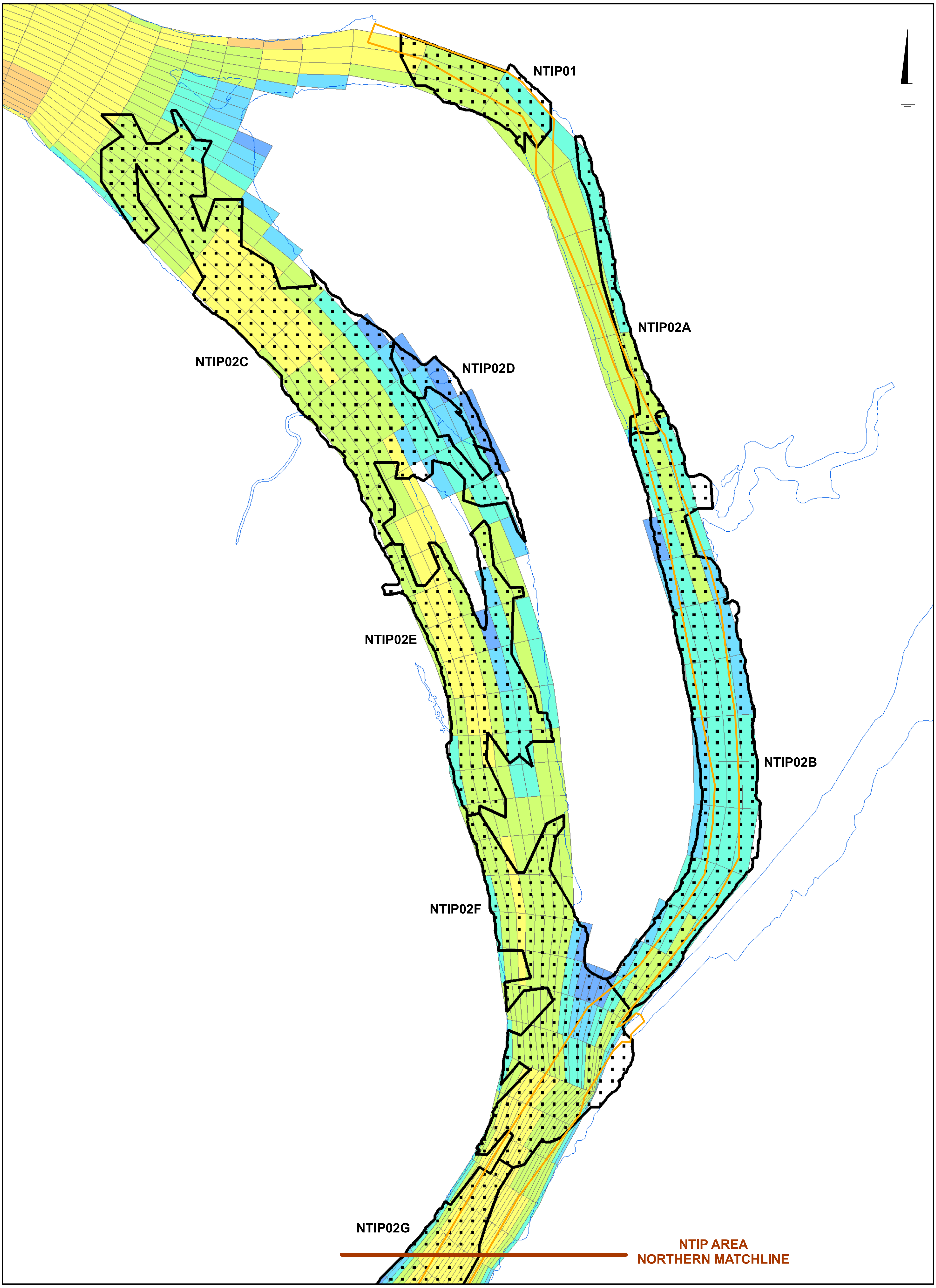
Figures



LEGEND:

- NAVIGATION CHANNEL
- DREDGE AREAS - REVISED BY BBL, INC.
- SHORELINE

VELOCITY - 10 YR (ft/s)

- 0 - 1
- 1 - 2
- 2 - 3
- 3 - 4
- 4 - 5
- 5 - 6
- 6 - 6.5

0 420 840 Feet

GRAPHIC SCALE

NOTE:

1. BASEMAPPING AND DATA PROVIDED BY QEA.
2. ESTIMATED LOCATION OF NAVIGATION CHANNEL AS DEVELOPED BY QEA AND BBL BASED ON HISTORICAL MAPS, DREDGING, AND BATHYMETRIC DATA.
3. SHORELINE AND SHORELINE STRUCTURES BASED ON AERIAL MAPPING PERFORMED BY CHAS H. SELLS, INC IN THE SPRING OF 2002.

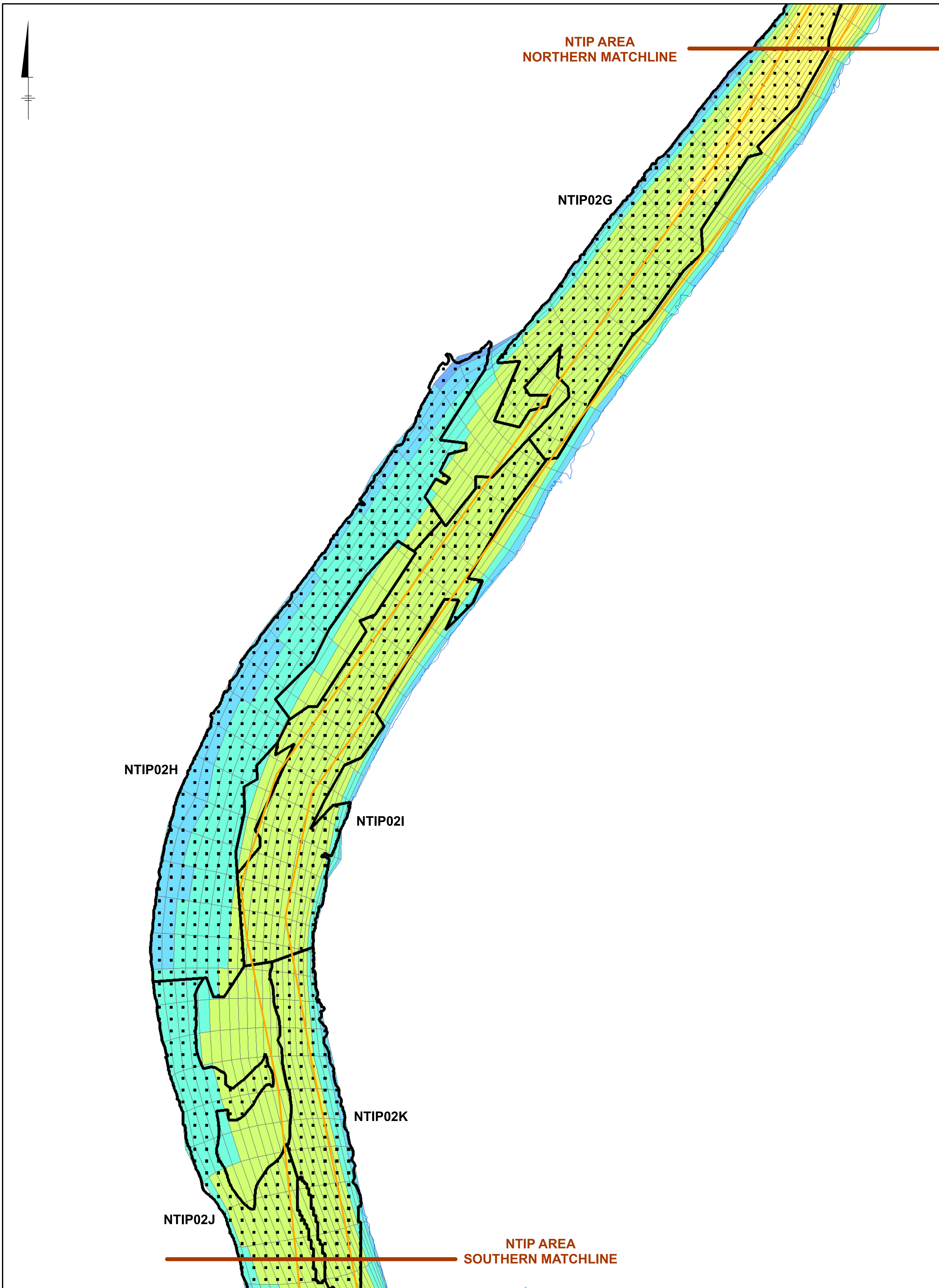
GENERAL ELECTRIC COMPANY
HUDSON RIVER PCBs SUPERFUND SITE
ATTACHMENT H - DESIGN ANALYSIS: BACKFILLING/CAPPING

DREDGE PRISMS SHOWING 10 YEAR VELOCITY DATA - NTIP NORTH

BBL
BLASLAND, BOUCK & LEE, INC.
engineers, scientists, economists

FIGURE
H1

8/20/05 SYR-85 MTK
GE Hudson (20437.800)
G:\GE_GIS\GE-HudsonRiver\VelocityShearStress_Phase1Areas\mxd\10 Year Velocity and SS - NTIP North 11x17.mxd



LEGEND:

- NAVIGATION CHANNEL
- DREDGE AREAS - REVISED BY BBL, INC.
- SHORELINE

VELOCITY - 10 YR (ft/s)

- 0 - 1
- 1 - 2
- 2 - 3
- 3 - 4
- 4 - 5
- 5 - 6
- 6 - 6.5

0 420 840 Feet

GRAPHIC SCALE

NOTE:

1. BASEMAPPING AND DATA PROVIDED BY QEA.
2. ESTIMATED LOCATION OF NAVIGATION CHANNEL AS DEVELOPED BY QEA AND BBL BASED ON HISTORICAL MAPS, DREDGING, AND BATHYMETRIC DATA.
3. SHORELINE AND SHORELINE STRUCTURES BASED ON AERIAL MAPPING PERFORMED BY CHAS H. SELLS, INC IN THE SPRING OF 2002.

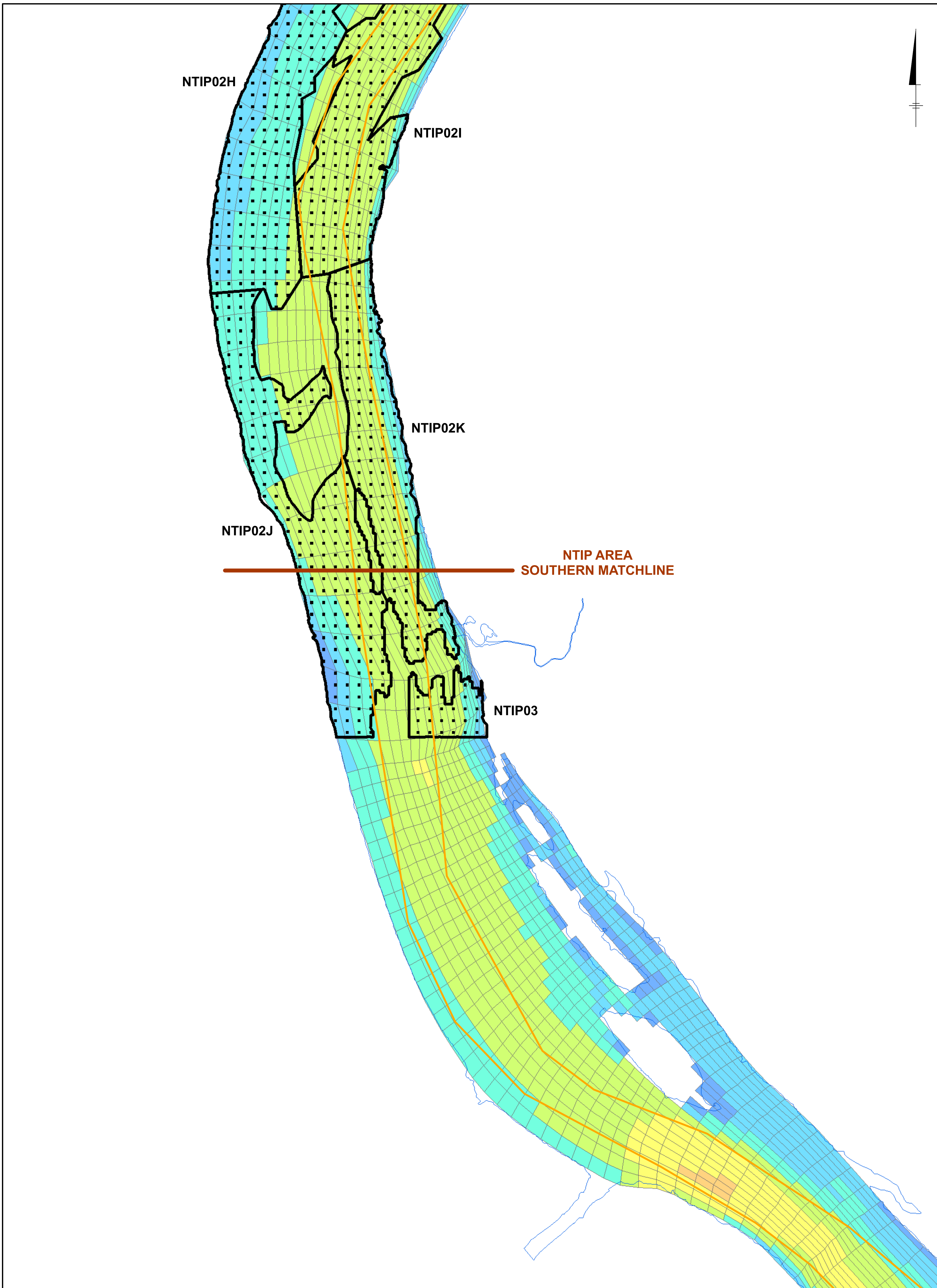
GENERAL ELECTRIC COMPANY
HUDSON RIVER PCBs SUPERFUND SITE
ATTACHMENT H - DESIGN ANALYSIS: BACKFILLING/CAPPING

DREDGE PRISMS SHOWING 10 YEAR VELOCITY DATA - NTIP CENTRAL

BBL® BLASLAND, BOUCK & LEE, INC. *engineers, scientists, economists*

FIGURE **H2**

8/20/05 SYR-85 MTK
GE Hudson (20437.800)
G:\GE_GIS\GE-HudsonRiver\VelocityShearStress_Phase1Areas\mxd\10 Year Velocity and SS - NTIP Central 11x17.mxd



LEGEND:

NAVIGATION CHANNEL

DREDGE AREAS - REVISED BY BBL, INC.

SHORELINE

0 420 840 Feet

GRAPHIC SCALE

VELOCITY - 10 YR (ft/s)

0 - 1

1 - 2

2 - 3

3 - 4

4 - 5

5 - 6

6 - 6.5

NOTE:

1. BASEMAPPING AND DATA PROVIDED BY QEA.
2. ESTIMATED LOCATION OF NAVIGATION CHANNEL AS DEVELOPED BY QEA AND BBL BASED ON HISTORICAL MAPS, DREDGING, AND BATHYMETRIC DATA.
3. SHORELINE AND SHORELINE STRUCTURES BASED ON AERIAL MAPPING PERFORMED BY CHAS H. SELLS, INC IN THE SPRING OF 2002.

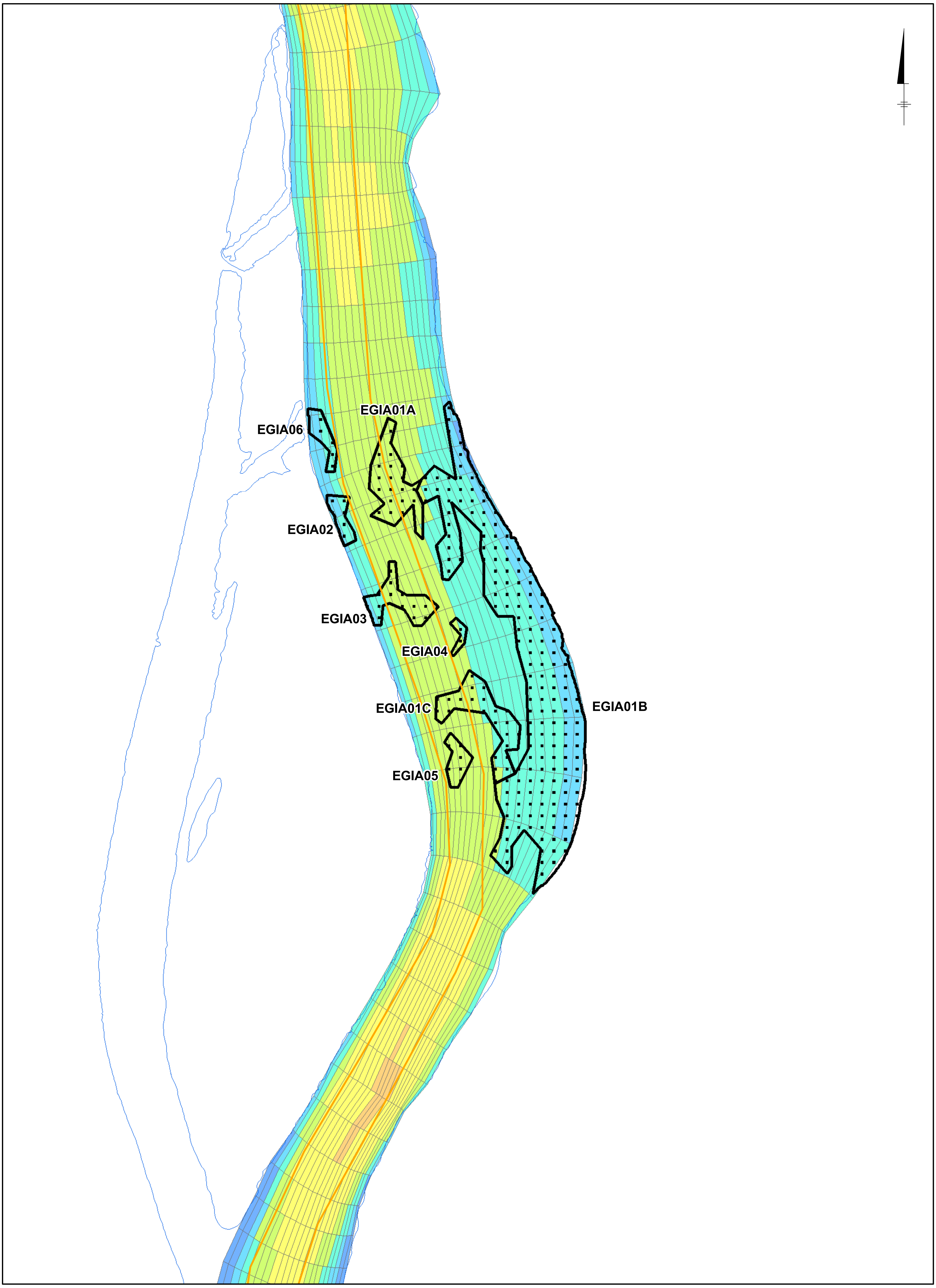
8/20/05 SYR-85 MTK
GE Hudson (20437.800)
G:\GE_GIS\GE-HudsonRiver\VelocityShearStress_Phase1Areas\mxd\10 Year Velocity and SS - NTIP South 11x17.mxd

GENERAL ELECTRIC COMPANY
HUDSON RIVER PCBs SUPERFUND SITE
ATTACHMENT H - DESIGN ANALYSIS:
BACKFILLING/CAPPING

DREDGE PRISMS SHOWING 10 YEAR VELOCITY DATA - NTIP SOUTH

BBL®
BLASLAND, BOUCK & LEE, INC.
engineers, scientists, economists

FIGURE
H3



LEGEND:

NAVIGATION CHANNEL

DREDGE AREAS - REVISED BY BBL, INC.

SHORELINE

VELOCITY - 10 YR (ft/s)

0 - 1

1 - 2

2 - 3

3 - 4

4 - 5

5 - 6

6 - 6.5

0 420 840 Feet

GRAPHIC SCALE

NOTE:

1. BASEMAPPING AND DATA PROVIDED BY QEA.
2. ESTIMATED LOCATION OF NAVIGATION CHANNEL AS DEVELOPED BY QEA AND BBL BASED ON HISTORICAL MAPS, DREDGING, AND BATHYMETRIC DATA.
3. SHORELINE AND SHORELINE STRUCTURES BASED ON AERIAL MAPPING PERFORMED BY CHAS H. SELLS, INC IN THE SPRING OF 2002.

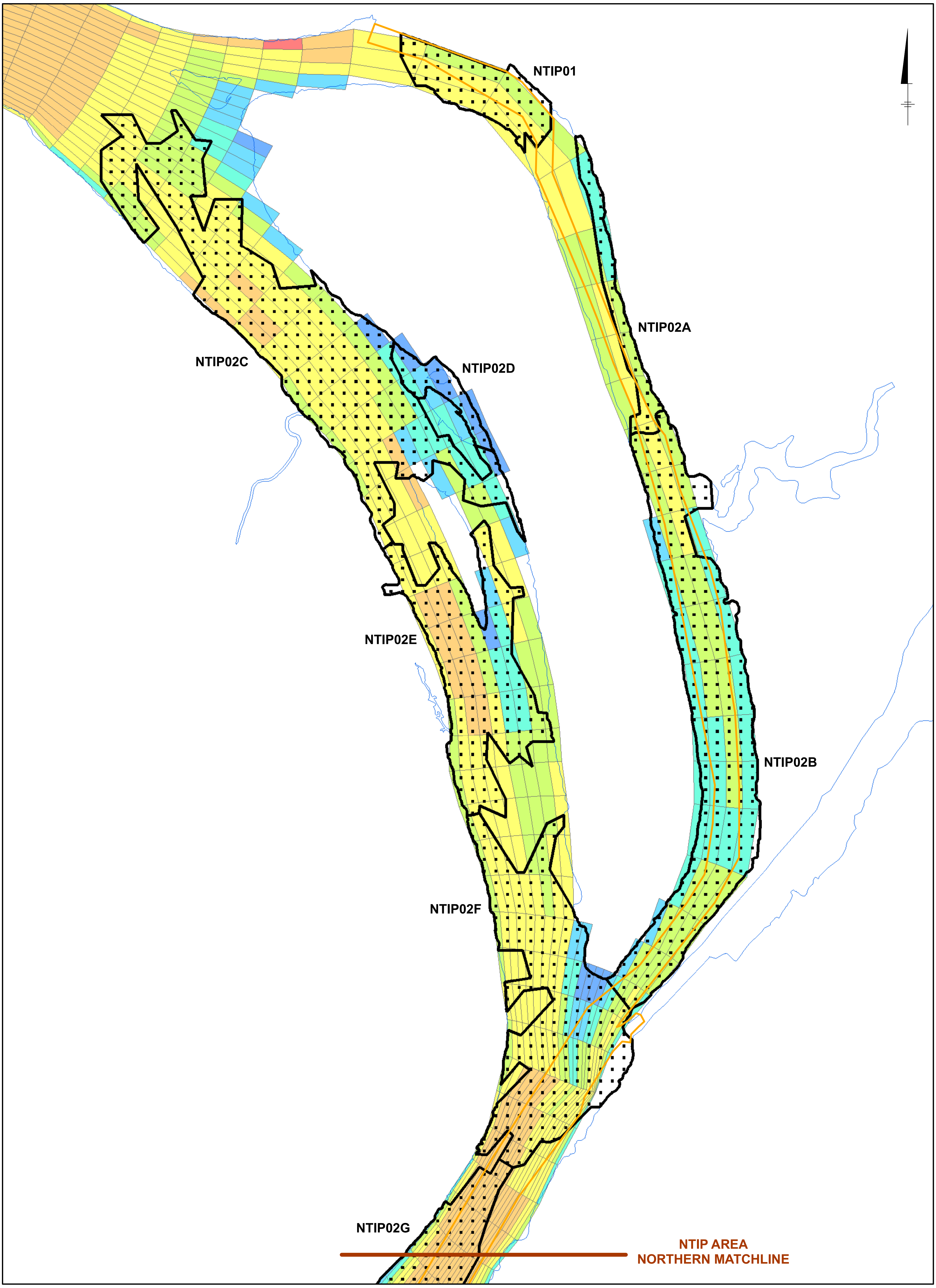
8/20/05 SYR-85 MTK
GE Hudson (20437.800)
G:\GE_GIS\GE-HudsonRiver\VelocityShearStress_Phase1Areas\mxd\10 Year Velocity and SS - EGIA 11x17.mxd

GENERAL ELECTRIC COMPANY
HUDSON RIVER PCBs SUPERFUND SITE
ATTACHMENT H - DESIGN ANALYSIS: BACKFILLING/CAPPING

DREDGE PRISMS SHOWING 10 YEAR VELOCITY DATA - EGIA

BBL BLASLAND, BOUCK & LEE, INC.
engineers, scientists, economists

FIGURE
H4



LEGEND:

NAVIGATION CHANNEL

DREDGE AREAS - REVISED BY BBL, INC.

SHORELINE

0 420 840 Feet

GRAPHIC SCALE

VELOCITY - 100 YR (ft/s)

- 0 - 1
- 1 - 2
- 2 - 3
- 3 - 4
- 4 - 5
- 5 - 6
- 6 - 6.5

NOTE:

1. BASEMAPPING AND DATA PROVIDED BY QEA.
2. ESTIMATED LOCATION OF NAVIGATION CHANNEL AS DEVELOPED BY QEA AND BBL BASED ON HISTORICAL MAPS, DREDGING, AND BATHYMETRIC DATA.
3. SHORELINE AND SHORELINE STRUCTURES BASED ON AERIAL MAPPING PERFORMED BY CHAS H. SELLS, INC IN THE SPRING OF 2002.

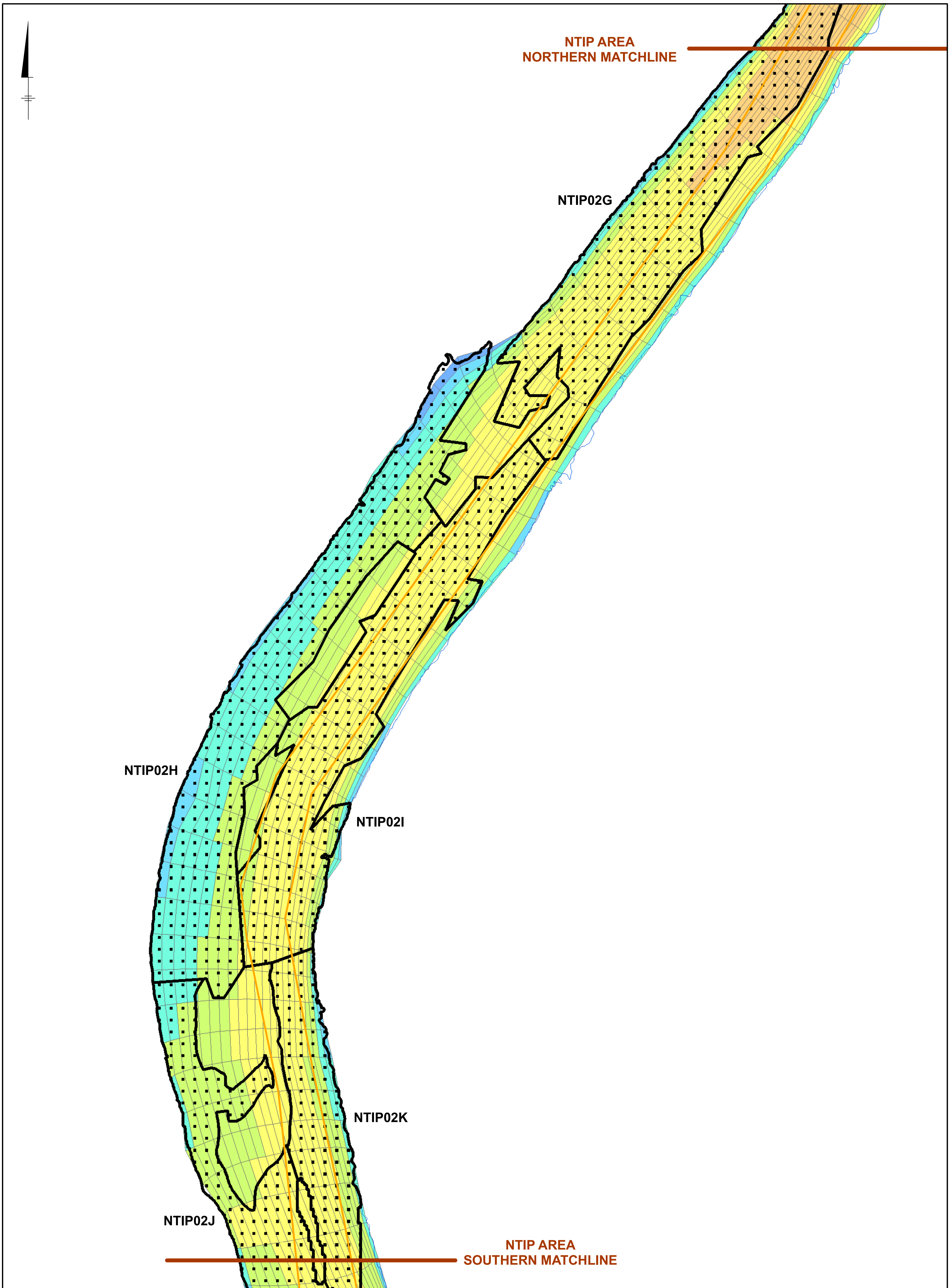
GENERAL ELECTRIC COMPANY
HUDSON RIVER PCBs SUPERFUND SITE
ATTACHMENT H - DESIGN ANALYSIS: BACKFILLING/CAPPING

DREDGE PRISMS SHOWING 100 YEAR VELOCITY DATA - NTIP NORTH

BBL® BLASLAND, BOUCK & LEE, INC. *engineers, scientists, economists*

FIGURE **H5**

8/20/05 SYR-85 MTK
GE Hudson (20437.800)
G:\GE_GIS\GE-HudsonRiver\VelocityShearStress_Phase1Areas\mxd\100 Year Velocity and SS - NTIP North 11x17.mxd



LEGEND:

- NAVIGATION CHANNEL
- DREDGE AREAS - REVISED BY BBL, INC.
- SHORELINE

VELOCITY - 100 YR (ft/s)

- 0 - 1
- 1 - 2
- 2 - 3
- 3 - 4
- 4 - 5
- 5 - 6
- 6 - 6.5

0 420 840 Feet

GRAPHIC SCALE

NOTE:

1. BASEMAPPING AND DATA PROVIDED BY QEA.
2. ESTIMATED LOCATION OF NAVIGATION CHANNEL AS DEVELOPED BY QEA AND BBL BASED ON HISTORICAL MAPS, DREDGING, AND BATHYMETRIC DATA.
3. SHORELINE AND SHORELINE STRUCTURES BASED ON AERIAL MAPPING PERFORMED BY CHAS H. SELLS, INC IN THE SPRING OF 2002.

GENERAL ELECTRIC COMPANY
HUDSON RIVER PCBs SUPERFUND SITE
ATTACHMENT H - DESIGN ANALYSIS: BACKFILLING/CAPPING

DREDGE PRISMS SHOWING 100 YEAR VELOCITY DATA - NTIP CENTRAL

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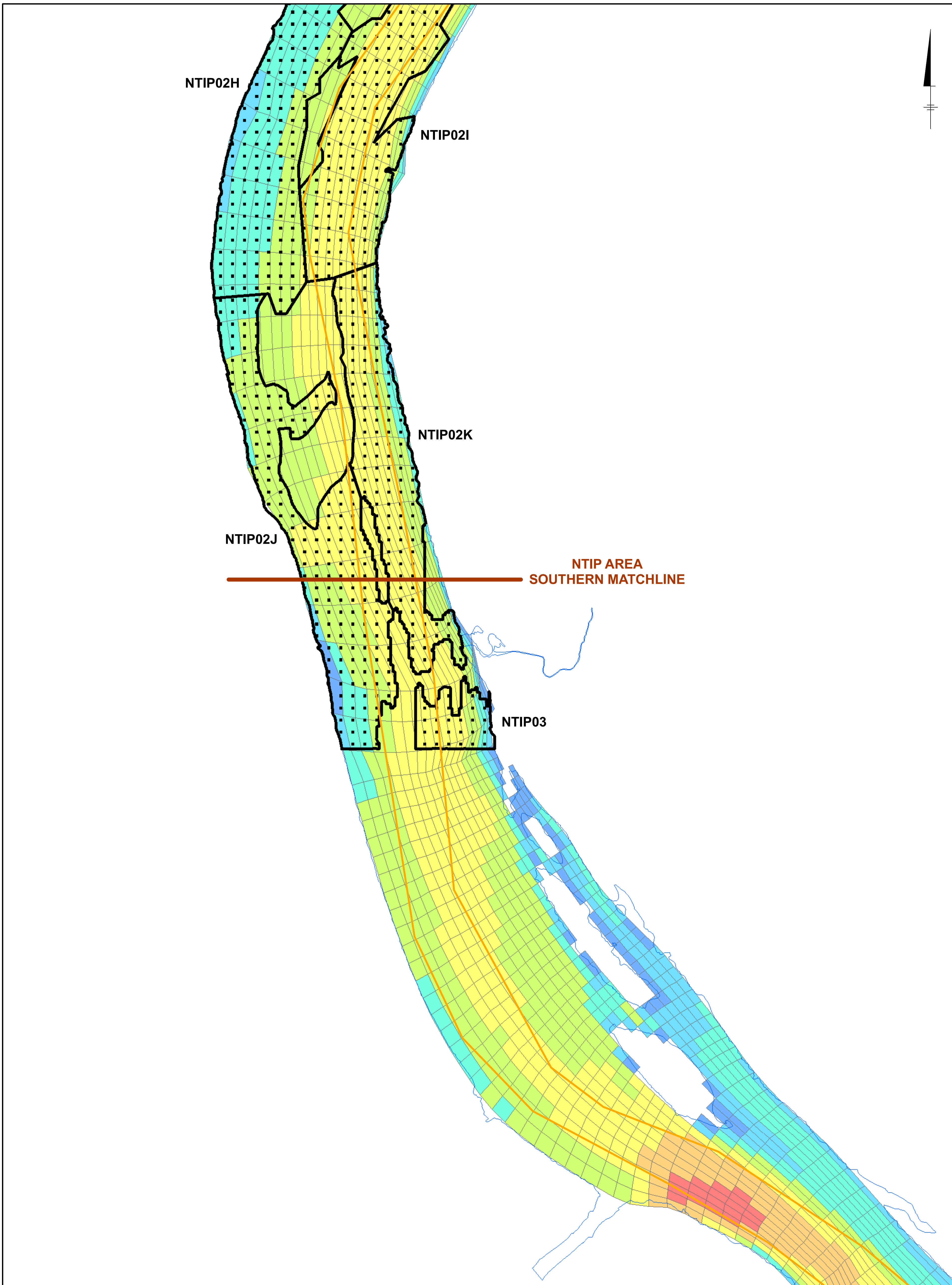
FIGURE
H6

8/20/05 SYR-85 MTK
GE Hudson (20437.800)
G:\GE_GIS\GE-HudsonRiver\VelocityShearStress_Phase1Areas\mxd\100 Year Velocity and SS - NTIP Central 11x17.mxd



LEGEND:

- NAVIGATION CHANNEL
- DREDGE AREAS - REVISED BY BBL, INC.
- SHORELINE

VELOCITY - 100 YR (ft/s)

- 0 - 1
- 1 - 2
- 2 - 3
- 3 - 4
- 4 - 5
- 5 - 6
- 6 - 6.5

0 420 840 Feet

GRAPHIC SCALE

NOTE:

1. BASEMAPPING AND DATA PROVIDED BY QEA.
2. ESTIMATED LOCATION OF NAVIGATION CHANNEL AS DEVELOPED BY QEA AND BBL BASED ON HISTORICAL MAPS, DREDGING, AND BATHYMETRIC DATA.
3. SHORELINE AND SHORELINE STRUCTURES BASED ON AERIAL MAPPING PERFORMED BY CHAS H. SELLS, INC IN THE SPRING OF 2002.

GENERAL ELECTRIC COMPANY
HUDSON RIVER PCBs SUPERFUND SITE
ATTACHMENT H - DESIGN ANALYSIS: BACKFILLING/CAPPING

DREDGE PRISMS SHOWING 100 YEAR VELOCITY DATA - NTIP SOUTH

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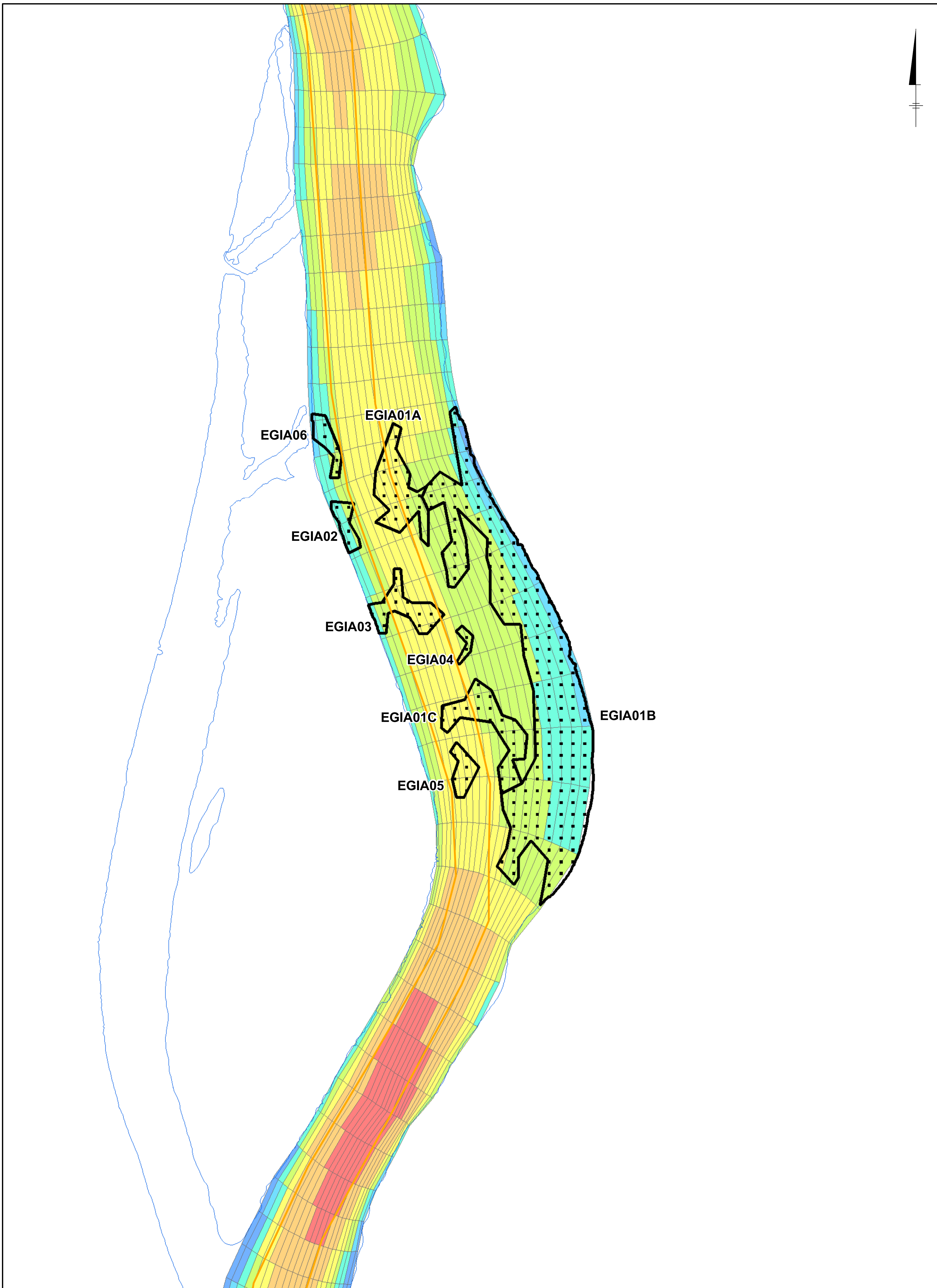
FIGURE
H7

8/20/05 SYR-85 MTK
GE Hudson (20437.800)
G:\GE_GIS\GE-HudsonRiver\VelocityShearStress_Phase1Areas\mxd\100 Year Velocity and SS - NTIP South 11x17.mxd



LEGEND:

NAVIGATION CHANNEL

DREDGE AREAS - REVISED BY BBL, INC.

SHORELINE

VELOCITY - 100 YR (ft/s)

	0 - 1
	1 - 2
	2 - 3
	3 - 4
	4 - 5
	5 - 6
	6 - 6.5

0 420 840 Feet

GRAPHIC SCALE

NOTE:

1. BASEMAPPING AND DATA PROVIDED BY QEA.
2. ESTIMATED LOCATION OF NAVIGATION CHANNEL AS DEVELOPED BY QEA AND BBL BASED ON HISTORICAL MAPS, DREDGING, AND BATHYMETRIC DATA.
3. SHORELINE AND SHORELINE STRUCTURES BASED ON AERIAL MAPPING PERFORMED BY CHAS H. SELLS, INC IN THE SPRING OF 2002.

8/20/05 SYR-85 MTK
GE Hudson (20437.800)
G:\GE_GIS\GE-HudsonRiver\VelocityShearStress_Phase1Areas\mxd\100 Year Velocity and SS - EGIA 11x17.mxd

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ATTACHMENT H - DESIGN ANALYSIS: BACKFILLING/CAPPING

DREDGE PRISMS SHOWING 100 YEAR VELOCITY DATA - EGIA

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FIGURE
H8