Flow Consolidation at Cottonwood Ranch
Platte River, NE

Feasibility Study Report

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EXECUTIVE SUMMARY

The Cottonwood Ranch (CTWR) habitat complex is located in the Overton to Elm Creek bridge segment, encompasses approximately four miles of river channel and includes lands owned by the Nebraska Public Power District and Platte River Recovery Implementation Program. The Platte River includes three distinct channels through the complex, the North, Main, and South channel. Currently flow is split among the 3 channels, with the North conveying 10%, the Main 66%, and the South 24% of the modeled 8,000 ft^3/s flow. Flow consolidation on the Platte River at the Cottonwood Ranch property endeavors to achieve a minimum of 85% of the volume at the 8,000 ft^3/s flow event in the Main channel, or 6,800 ft^3/s, utilizing only water from the South channel.

The approach to flow consolidation focused on the use of hydraulic controls in the South channel to push water through an overflow channel into the Main channel. Downstream un-consolidation of flow appears to be accomplished easily as water occupies natural flow paths just below the CTWR boundary. Upstream hydraulic implications were also a concern and intended to be minimized. A HEC-Geo RAS model was available for evaluating the hydraulic characteristics of various flow scenarios. A coarse review of the hydraulic model indicated that observed results from USGS gages and field data did not match exactly with predicted results from the model. The magnitude of the differences varied with flow, but the model was still deemed appropriate for evaluation purposes.

A total of five scenarios were applied to the model, each building upon the other to arrive at a solution that fulfilled the flow criteria for consolidation. Two scenarios that combined a significant hydraulic control on the South channel with an overflow channel to the Main channel were successful in achieving the minimum flow criteria. The location of the overflow channel varied in each scenario, from a more upstream location to a location further downstream, and approximately in the middle of the CTWR property. The latter location had some implications for hydraulic changes to an adjacent hay field on the left bank of the Main channel that may require further investigation if that option is pursued.

The hydraulic controls if constructed on site, are intended to use natural materials that mimic the natural features present in the Platte. The first option is a sand plug which would act to back water in the South channel and force it down the overflow channel into the Main channel. Although inexpensive to build, they are more prone to wash out and may inhibit low flows to a greater degree than the second option, a channel spanning log jam. The log jam would mimic a beaver dam to some extent, allowing low flows to pass through while obstructing high flows and creating the head to drive flow from the South into the Main channel. The log jams are more expensive to build, but are a more permanent hydraulic control than the sand plug. Minor hydraulic controls would need to be constructed on a number of overflow channels, and would likely feature a combination of sand, logs, and vegetation to create an efficient flow barrier.

It appears from the data that flow consolidation may be evolving already on the CTWR site, as USGS gages predict more flow in the Main channel than is currently shown in the model. This coupled with the initiatives for Sediment Augmentation and Short Duration High Flow events, may accomplish flow consolidation without any manipulation of the South channel. Still, if consolidation were to be accomplished the approach is recommended to occur under two phases. The first phase would be a pilot phase and feature the excavation of an overflow channel in an appropriate location, followed by the construction of a sand plug or plugs in the South channel to act as a major hydraulic control. Additional minor hydraulic controls would be constructed in overflow channels in both the South and Main channels, to ensure water was captured efficiently. Culverts to ensure low flow conveyance can be added to the sand plug as needed. This project should be monitored over the course of a SDHF event and adjusted until the outcome is deemed acceptable. At this point, Phase 2 could be constructed using log jam structures for the major hydraulic control, creating a more permanent and potentially longer lived solution if cottonwood and willow stakes can be incorporated to create a "living jam" that would persist in the environment of the South channel.

BACKGROUND

Recovery Effort

The Platte River Recovery Implementation Program (PRRIP or "Program") is in the process of implementing two strategies for recovery of the historic habitat on the river. The Mechanical Creation and Maintenance strategy endeavors to manually create and indefinitely maintain habitat meeting the specific life-history needs of federally listed whooping crane, piping plover, and interior least tern. The Flow-Sediment-Mechanical (FSM) strategy seeks to create this same habitat through the manipulation of existing vegetation regimes, flood pulses, and sediment loads. A detailed discussion of these exists within the *Adaptive Management Plan* section of the document *Final Platte River Recovery Implementation Program* (2006).

This effort is focused on the Mechanical component of the FSM approach which will consolidate in the main channel 85% or more of the volume of flow during events in the 6000-8000 ft^3/s range. Flow consolidation will increase stream power within a target corridor width of 750' – 800', reworking sand bars and theoretically reinstating braided channel processes within the new hydrologic disturbance regime.

The objective of this study is to identify reasonable approaches to accomplish flow consolidation at approximately 8,000 ft^3/s through the Cottonwood Ranch (CTWR) property into the existing main channel within a flow corridor of approximately 750'.

Guiding Criteria

Alternatives for consolidation, and un-consolidation, of flow through the CTWR property will be evaluated using criteria below developed from discussions and experience of both Program Staff and Inter-Fluve.

- Off property effects, both above and below CTWR, should be eliminated or minimized
- Consolidation should be affected by adding flow volume to the Main channel from the South channel
- Un-consolidation should be affected by adding flow from the Main channel back to the South channel thereby minimizing any downstream effects
- A minimum of 85% of the 8,000 ft^3/s discharge (6,800 ft^3/s) should be consolidated in the Main channel
- The target wetted width of the Main channel at 8,000 ft^3/s following consolidation is 750-800'
- Natural approaches are preferred over rigid, engineered water control structures
- Low cost, low maintenance solutions are preferred

- Solutions should minimize disruption to both the ecological and physical function that has developed in the South channel
- The North channel will not be modified to route water into the Main channel

EXISTING SITE CONDITIONS AND INFORMATION

The CTWR habitat complex occupies approximately 4.0 miles of the main channel of the Platte beginning just downstream of the Overton Bridge (Figure 1). The river through this section is comprised of three distinct channels; a North, Main (middle), and South channel. As mentioned above, the focus of this study is pushing water from the South channel into the Main channel. No work will occur within the North channel.

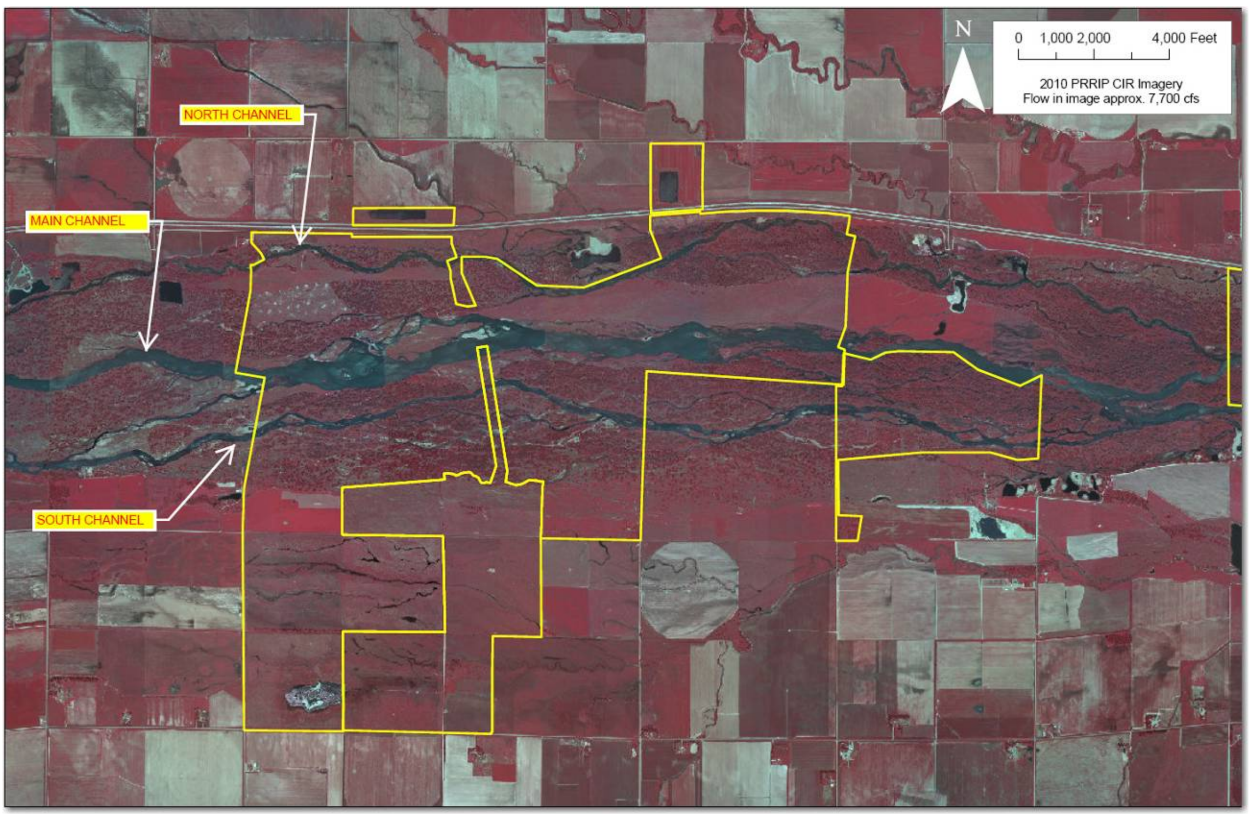

Figure 1: Aerial view of the CTWR property

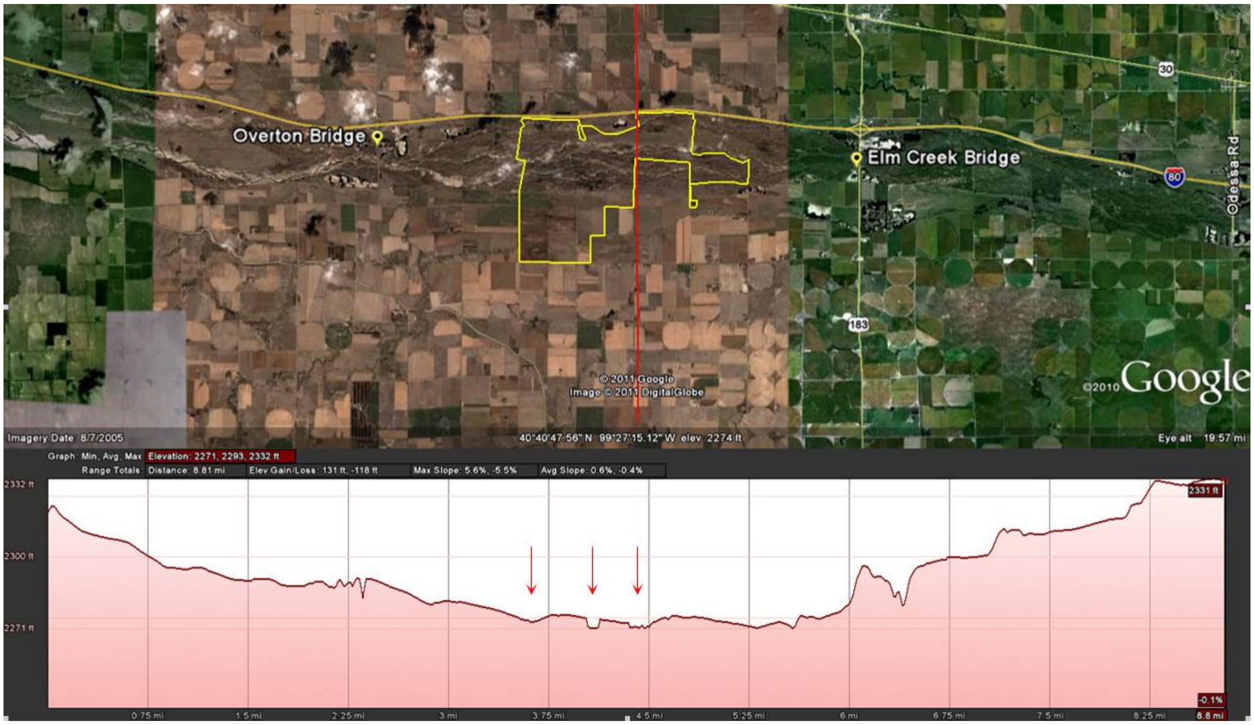

Figure 2: Valley cross section through the CTWR complex. The arrows indicate (left to right) the North, Main and South channels of the Platte

Contemporary Geomorphology

The Platte at CTWR flows within a valley approximately 3.0 miles wide, depending on the boundaries delineated. A rough valley profile using *Google Earth* (Figure 2 above) clearly shows the North, Main and South channels, and indicates the cross section of the valley slopes slightly to the south. The river is consolidated into a single channel as it passes under the Overton bridge, upstream of the CTWR property. Shortly after, the river splits into three distinct channels, the Main and South described briefly below. Downstream of the CTWR property, the river re-consolidates as it passes under the Elm Creek bridge. The stretch below Elm Creek bridge, in particular, below the Kearney Canal, represents a modern analog of conditions desired at the CTWR property. This pattern is maintained downstream until the Odessa bridge, where the channel takes on a more anabranch pattern, similar to what is seen at CTWR.

Figure 3: The Main channel on the CTWR property

The Main channel, the largest of the three, still maintains the sand bars and unstable banks that are the trade marks of a braided river, but these features are weakly defined. The bed and banks are composed of sand, but some deposits of much coarser material are noted on bars, along with vegetation. A layer of slightly silty material was noted within the right bank of the Main channel that seemed to serve as an aquatard, keeping flow in the Main channel from moving to the South through subsurface flowpaths. The extent and origin of this material was not investigated in detail, though it may exert influence in regulating the exchange of sub surface water between the Main and South channels. This issue is developed further, later in the report.

The South channel at higher flows appears as an anabranch planform, with a number of flow paths within the South channel and between the South and Main channels evident. At flows observed in the field, around 4,000 ft^3/s most water is maintained within a single, sinuous thread, set within defined banks. Channel bars are evident within the South channel indicating active sediment transport, likely from both upstream and channel margin sources.

Figure 4: The South channel on the CTWR property

Hydrology

Each of the three channels currently conveys a portion of the active flow during an 8,000 ft^3/s event through the CTWR property. Based on results from the hydraulic model (See Appendix A) the North channel conveys 10% (800 ft^3/s), the Main channel 66% (5,300 ft^3/s), and the South channel 24% (1,900 ft^3/s) under a modeled flow of 8,000 ft^3/s. Based on model results, to consolidate a minimum 85% of the 8,000 ft^3/s event in the main channel, the target flow is 6,800 ft^3/s, requiring the transfer of at least 1,500 ft^3/s from the South channel. Three USGS gages are present within close proximity to the CTWR property (Figure 5) providing both stage and discharge information for various events on both the Main and South channels. These gages indicate that more flow may be present in the Main channel, and less in the South, than is currently predicted in the model. A more detailed discussion on this topic can be found in Appendix A.

Figure 5: USGS gages located on or near the CTWR property

Previous work by the Bureau of Reclamation (Sanders, 2001) indicates that groundwater flow through the Central Platte strongly follows the downstream direction of the valley, as water enters laterally from higher elevations both north and south of the valley. Further, the report indicates groundwater is typically higher than the surface elevation of the Platte river several thousand feet perpendicular to the river corridor. Thus, groundwater hydrology observed as surface water elevations across the channels of the CTWR property are of particular interest to this project due to the *surface* and *subsurface* hydraulic gradients that would be manipulated to reach the goal of flow consolidation. During an April 2011 field visit by the project team, three cross sections were surveyed to compare relative water surface elevations in the Main and South channels (Figure 6).

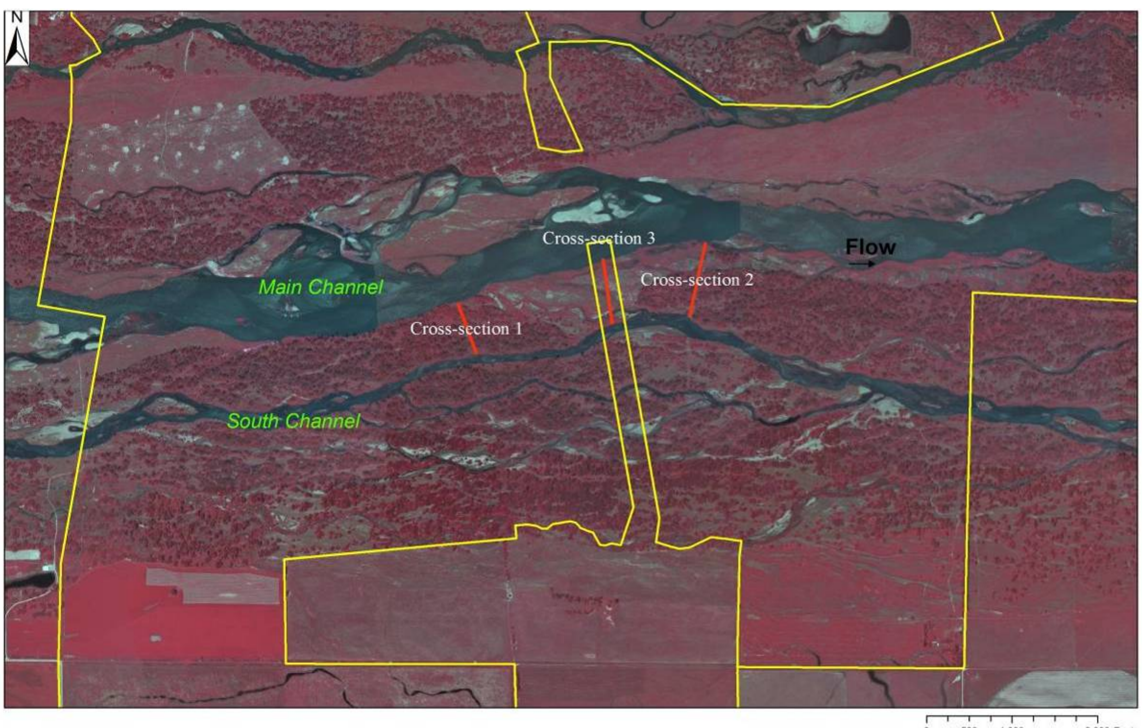

Figure 6: Cross sections surveyed on 4/7/11 at CTWR

Flow on the day of survey was about 3700 ft^3/s at the Overton gage, and remained steady throughout the day. Table 1 illustrates the difference between water surface elevations (WSE) between the two channels at each cross section. In all cases the elevation of the Main channel remained substantially higher than the South channel. The magnitude of the difference was somewhat surprising, but was illustrated distinctly at a location along the right bank of the Main channel where deposited fill overlying a native floodplain soil created a hydraulic differential of 1-2' across a horizontal distance of not more than 5' (Figure 7).

Table 1: Difference in WSE (water surface elevation) between the Main and South channels

Survey Section	Difference in WSE (Main – South)
1	3.11'
2	2.47'
3	3.28'

Figure 7: Blocked entrance to channel taking flow from the Main stem toward the South channel. The difference between the water surface elevations is called out.

The native soil underlying the fill (Figure 8) included some component of fines and when textured the slight tackiness of clay and slick characteristics of silt were noted. The horizontal extent and depth of this soil layer was not explored in detail, and its role in acting as an effective aquatard can only be hypothesized at this point. The implications of this and other similar observations of the apparent ability of the sand soils within the project area to maintain substantial sub-surface water gradients is discussed later in this report, but in short, holds promise for utilizing sand and soil plugs to provide short term hydraulic controls on the site.

Figure 8: Soil layer evident along the Main channel acting as an aquatard to flow. Soil to the left of the dashed line is sand fill, the cohesive material can be seen just below the water surface to the right of the dashed line

Vegetation

Vegetation within the project area between Main and South channels is dominated by a mature cottonwood forest. These trees dominate a narrow band between the South and Main channels (Figure 9). The stand appears to be of similar age. Informal estimates of these trees place the average DBH around 20" and height 60-80' to the top of the canopy. Other dominant vegetation layers are a scrub/shrub layer composed of Russian Olive, Willow *sp.,* and Dogwood *sp.* as well as a herbaceous layer dominated by various sedges and Reedcanary grass.

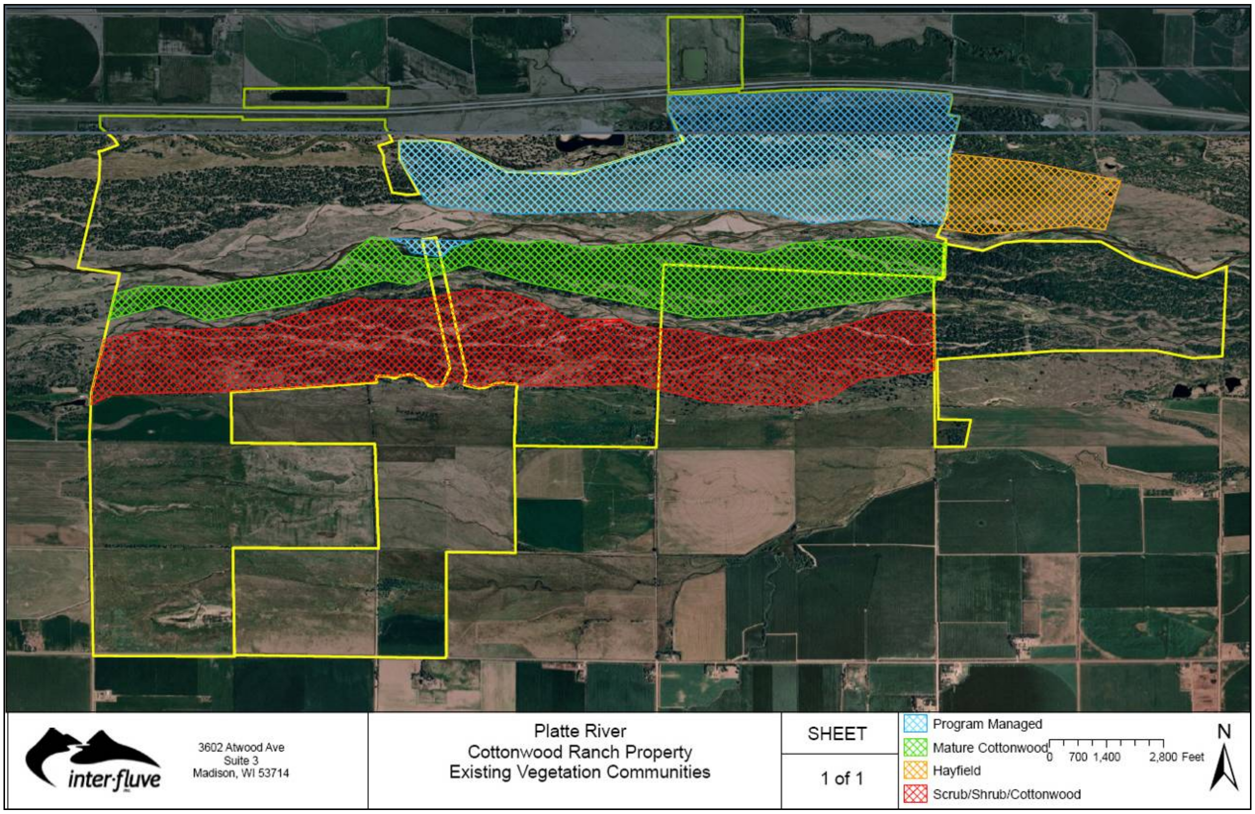

Figure 9: Observed vegetation regimes at CTWR

Hydraulic Model (HEC-RAS)

A detailed accounting of the HEC-RAS model and analysis is included in Appendix A. A short summary is included here to frame salient issues. The model was used for evaluating the hydraulic implications of various approaches to flow consolidation. Built for the Program in 2009, the model had undergone calibration using various data sources and included the complex (North, Main, South) channel morphology of the CTWR property. It was deemed appropriate for this feasibility level assessment of flow consolidation, though some calibration challenges became evident during analysis. In particular, when comparing model output to the flows and stages recorded by the three USGS gages on site (see Figure 5 above), the model under predicted the flow in the Main channel, and over predicted the flow in the South channel for a given total discharge. The reasons for this were unclear and beyond the scope of this effort, but are likely due in part to the shifting morphology of the Platte which can easily move large volumes of water between the Main and South channels as channel morphology shifts. The importance of this in evaluating the possibilities for flow consolidation are likely minor, as this effort is concerned with finding broad conclusions related to the efficacy of such an approach. The more important issue raised with calibration data is whether flow consolidation is occurring already on the site via natural processes. This is given greater attention in the *Recommendations* section of this report.

Adjacent Landowners

The Program operates under a "good neighbor policy" at CTWR as well as other managed lands. Among other things, this policy assures that management actions at CTWR should have minor or no consequences to adjacent landowners.

The area which encompasses the South channel upstream of the CTWR property is owned by a private individual. The ford used by vehicles to cross the South channel lies just upstream of the CTWR property line. A cabin structure also on this property is located upstream of the CTWR property. No other structures, aside from goose blinds, reside within an area above CTWR that might be sensitive to flow consolidation manipulation.

Downstream of the CTWR property on the left bank of the Main channel is an actively cut hay field. There are no structures on this property that would be sensitive to flow consolidation approaches, though increasing flood elevations on the hay field itself should be avoided. Further, all ground downstream of CTWR on right bank of the Main channel and including the South channel appears to be unmanaged riparian floodplain without any structures that might be effected by flow consolidation.

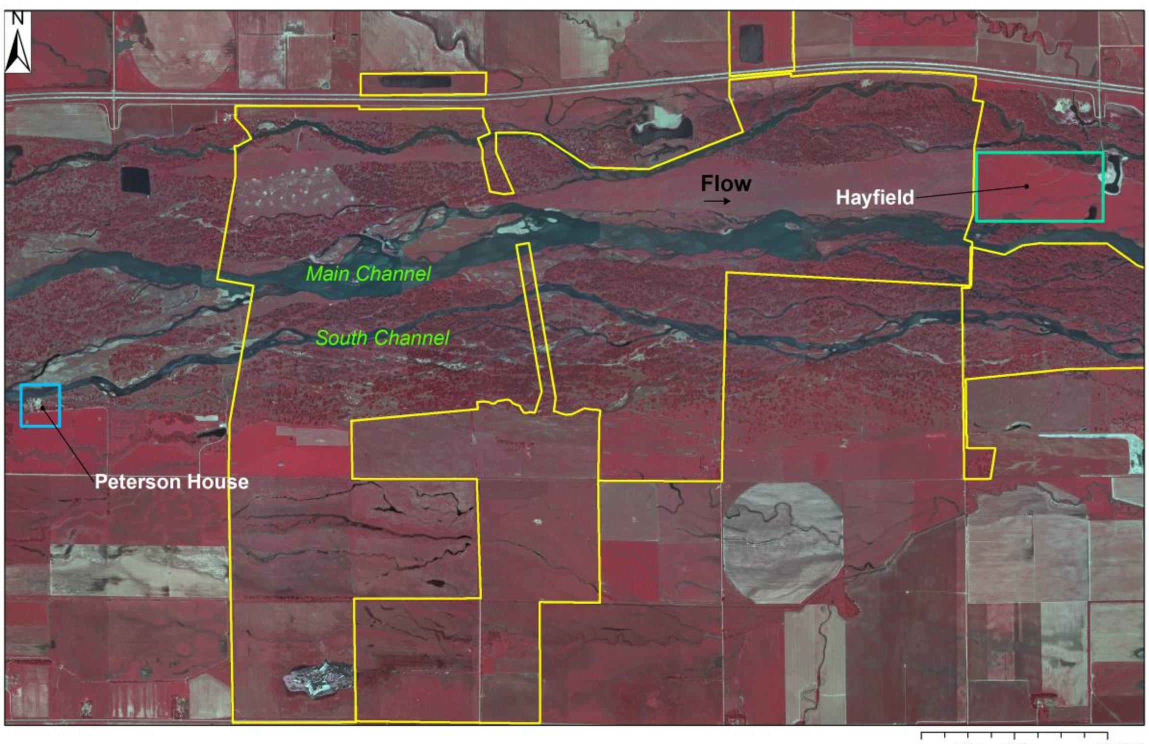

Figure 10: Adjacent structures and lands of interest to the CTWR property

ALTERNATIVES ANALYSIS

Analysis Approach

Approaches for flow consolidation focused on moving water from the South channel into the Main channel using hydraulic controls. From the outset, hydraulic controls using natural materials and methods that worked in tandem with the processes of erosion and deposition on the Platte were pursued. The cost, long term maintenance, and static nature of traditional engineered structures (concrete levees, dams, etc) immediately excluded this approach from analysis. Examples of natural methods include the use of sand plugs, large woody debris jams, roughness elements (high density vegetation or the use of smaller log jam structures), and existing or augmented topographic features on the site. All of these options accomplish the same result as traditional engineered methods, moving water around on the CTWR site. Further, these natural, yet engineered, options provide a low cost and effective means for a build – observe – adjust approach to implementation. If a sand plug or log jam is breached in a flood, the consequences are minor compared to the abandonment of a concrete weir or similar "hard" engineered structure. Inter-Fluve, NPPD, and the Program all have experience using natural materials on the Platte and other rivers that was brought to bear on the practical (qualitative) aspects of training water and preserving habitat in the Platte. Supplemental to this qualitative experience, the existing HEC-RAS model was used to evaluate the hydraulic implications of different scenarios and provide a quantitative test for whether flow criteria were met. Appendix B includes a detailed discussion of the hydraulic modeling effort and the results are summarized below.

In addition to the criteria mentioned above, this evaluation was searching for a fatal flaw that indicated attempts at consolidation would have a high probability for failure or would be prohibitive in some other way (cost, permitting, constructability etc). The analysis of river environments, particularly those as dynamic as the Platte, can at best provide conclusions on trends or probabilities for success. There are no absolute guarantees, a fact borne out in the well organized adaptive management approach the Program has pursued to date in restoration efforts on the Platte. It is important to state explicitly that the sand bed system of the Platte presents particular challenges to the long term persistence of any attempts at river training.

Hydraulic Scenarios

A total of five scenarios were evaluated on the site at the 8,000 ft^3/s event. A detailed discussion is included in Appendix B, but is summarized here. Each scenario built roughly upon the previous in an iterative approach. The hydraulic modeling confirms that an approach utilizing an overflow channel between the South and Main channel coupled with a major hydraulic control in the South channel will meet the design criteria for minimum flow consolidation. The location of this overflow channel does not appear to affect the outcome based on the modeling results.

SCENARIO	DESCRIPTION
1- Upstream Overflow Channel	*Summary* - included simply modeling a constructed overflow channel, dug between the South and Main channels near the upstream end of CTWR *Results* – the head differential was not great enough to push water through the overflow channel into the Main channel. Flow reported in the model was 150 ft^3/s well below the minimum flow of 1200 ft^3/s
2- Upstream Overflow Channel + Roughness	*Summary* – building on scenario 1, the roughness co-efficient was increased throughout the length of the South channel to simulate the addition of logs or large sand bedforms in an attempt to increase head and drive more water to the Main channel *Results* – the head differential was increased and the model reflected additional flow, 880 ft^3/s, in the overflow channel, though still below the minimum flow of 1200 ft^3/s
3- Upstream Overflow Channel + Hydraulic Control	*Summary* – building on scenario 2, the additional roughness elements were removed and instead an "inline structure" meant to simulate a hydraulic control was placed just downstream of the overflow channel in the South channel. *Results* – the more substantial structure created the head necessary to increase flow into the overflow channel. The model predicted 1340 ft^3/s, which exceeded the minimum flow criteria of 1200 ft^3/s.
4- Upstream Overflow Channel + Multiple Hydraulic Controls	*Summary* – though scenario 3 met the criteria for flow consolidation, the lowered water surface elevation downstream of the obstruction in the South channel caused concern for significant inflow back into the South channel. As a result, four hydraulic controls were added in the model (manifest as either a log jam or a sand plug) to maintain head and create a series of ponds. *Results* – the multiple hydraulic control structures maintained periodic pools interspersed with flow at lower volumes and lower elevations. The model indicated flow criteria for consolidation were still met.
5- Downstream Overflow Channel + Hydraulic Control	*Summary* – Moving the overflow channel further downstream along the South channel may provide several advantages. The overflow channel was moved downstream in the model and a single "inline structure"

5- Downstream Overflow Channel + Hydraulic Control (CON'T)	placed below to move water from the South into the Main channel. *Results* – The model predicted identical results to Scenario 3, indicating little difference in the predicted impact of locating the overflow channel in its original upstream location or in more downstream locale. Modeled flow in the overflow channel was 1300 ft^3/s, which met the minimum flow criteria of 1200 ft^3/s. Of note however, this scenario does increase flood elevations on the hay field along the left bank of the Main channel. The validity of this result should be considered with respect to the resolution with which the model can predict such elevations accurately.

Table 2: Summary of results in the hydraulic analysis of flow consolidation at CTWR

Overflow Channel Location

Locating the overflow channel further downstream on the property as opposed to upstream provides several advantages. A low water ford crossing exists just upstream of the CTWR boundary on the South channel. Used by the upstream landowner, this crossing is an important access route. Although the hydraulic model indicates no changes to water surface elevations would occur at this location, logical reasoning indicates that added roughness may induce deposition in the South channel that could migrate upstream. Locating the overflow channel further downstream minimizes this risk. As noted in the field by NPPD and Program staff, the downstream location allows a control section to be evaluated upstream through the CTWR property where no flow consolidation will occur. This may allow observations on the CTWR property to be separated into natural versus flow consolidation - induced changes in the Main channel.

The model does indicate that moving the overflow channel to this location increases flood elevations along the hay field just downstream of the property boundary. The increase is slight (0.5') and may be occurring already according to results from the USGS Main channel gage. If this option is pursued in final design, the reality of this predicted impact should be investigated further.

Sand vs. Wood Hydraulic Controls

Both sand and large wood (in the form of Cottonwood Trees) are readily available on site in numbers or volumes sufficient to construct flow controls efficiently and at relatively low cost. There are some consequences to failure of these controls. In the case of a sand plug, the material is simply added to the natural sediment load of the channel, with failure usually occurring when the plug is overtopped. In the case of large wood jams a complete failure, allowing all logs to move downstream could create a considerable maintenance issue at the Kearney diversion, a noted concern of NPPD staff. The potential for such a massive failure is low, as all logs would be cabled together to form a substantial, coherent structure. The failure mechanism of such a structure, if it occurs, may allow several logs of perhaps a structure composed of 50 logs to mobilize downstream, but a mass failure and transport of the entire structure is unlikely. Further, the bankfull width of the South channel coupled with the extensive riparian vegetation would make conditions for transport of even single trees downstream to perhaps the Kearney Diversion difficult, though not impossible.

Sand plugs in the channel will likely create a more dramatic damming effect at both low flows and design flows than wood jams. The addition of small culverts will allow low flow to pass through the sand plugs and maintain conveyance downstream. Wood jams should remain somewhat permeable to flow, particularly low flows, through the structure. Given the active sediment transport occurring in the South channel, sand will likely deposit within or upstream of any log jam structure, inhibiting flow through the structure after a few flood events. However, the elevation of this deposition should not completely fill the structure. The porosity of the log jam structure also creates a challenge in affecting the right amount of hydraulic control to achieve flow consolidation, whereas the sand plug, given that it is a solid structure, can be easily modeled and built to a predetermined elevation to ensure flow criteria will be met.

Figure 11: Sand dam at a mine near the CTWR site. WSE is about 4' higher on the left than on the right, with only minor seepage. Plugs like this can be effective at CTWR to move water from one channel to another.

Construction costs differ between the use of sand or logs for hydraulic control. The basic equipment required for each is the same, an excavator, haul truck, and bulldozer. Given that materials can be readily acquired on site, an approximation of cost based on construction days required to complete various components is useful. Table 3 provides unit costs; a simple means for assessing a wide range of approaches.

Table 3: Estimated unit quantities and costs for construction elements

	Sand Control	**Log Jam Control**
Construct 1 Major Control	0.5 days	5 days
Excavate Overflow Channel	2 days	2 days
Construct 1 Minor Control	0.25 days	1.5 days
Equipment and Operators	$3000 / day	
Engineer Oversight	$1200 / day	
TOTAL LABOR	**$4200 / day**	
Assumptions: • Each Major Log Jam is comprised of 50 trees • Each Minor Log Jam is comprised of 30 trees • Trees can be harvested on site at a rate of 10 /day • Engineer oversight assumes a 10 hour day • Material costs are negligible (cable, vegetation stock and seed)		

Applying the unit costs in Table 3 to hydraulic scenarios which met design criteria provides comparative costs, illustrated in Tables 4-6. For estimation purposes the ratio of minor hydraulic controls to major hydraulic controls is 2:1. Minor controls will be necessary to limit conveyance on small channels adjacent to the Main and South channels. Note, maintenance costs were not tabulated. Sand plugs are subject to failure when overtopped, and may require rebuilding. Log jams should require little maintenance.

SCENARIO 3 Overflow Channel + 1 Major Hydraulic Control	**Sand Control**	**Log Jam Control**
Construct 1 Major Control	0.5 days	5 days
Excavate Overflow Channel	2 days	2 days
Construct 2 Minor Controls	0.5 days	3 days
Subtotal @ $4200 / day	**3 days**	**10 days**
CONSTRUCTION ESTIMATE	**$12,600**	**$42,000**

Table 4: Approximate construction costs for flow consolidation under Scenario 3 CTWR

SCENARIO 4 Overflow Channel + 4 Major Hydraulic Controls	**Sand Control**	**Log Jam Control**
Construct 4 Major Controls	2 days	20 days
Excavate Overflow Channel	2 days	2 days
Construct 8 Minor Controls	2 days	12 days
Subtotal @ $4200 / day	**6 days**	**34 days**
CONSTRUCTION ESTIMATE	**$25,200**	**$142,800**

Table 5: Approximate construction costs for flow consolidation under Scenario 4 CTWR

SCENARIO 5 Downstream Overflow Channel + 1 Major Hydraulic Control	**Sand Control**	**Log Jam Control**
Construct 1 Major Control	0.5 days	5 days
Excavate Overflow Channel	2 days	2 days
Construct 2 Minor Controls	0.5 days	3 days
Subtotal @ $4200 / day	**3 days**	**10 days**
CONSTRUCTION ESTIMATE	**$12,600**	**$42,000**

Table 6: Approximate construction costs for flow consolidation under Scenario 5 CTWR

The persistence of the log jam controls constructed of Cottonwood trees in the Platte environment is a concern. Whereas sand dams are expected to regularly washout and be rebuilt, if designed correctly, a log jam structure should remain largely intact at the location of construction. This is not without its challenges however. The dynamics of a highly erodible sand bed channel make it difficult to construct anything of a permanent nature. Strategies that can be employed to overcome this include size of the log jam, extending the structure laterally well into the existing channel banks as well as vertically below a predicted scour depth. The use of live vegetation is a second stabilizing strategy, the root systems providing a matrix to reinforce the sand against scour.

Logs subjected to wetting and drying and a wide range of temperature and humidity regimes can quickly degrade as well. Specific information on the decay rate of cottonwood is not available, but a reasonable assumption for design purposes is 5-10 years for the overall coherence of the structure to be maintained before requiring maintenance due to decay. One approach to extend this design life is to use cuttings or bare root stock of both native willow and cottonwood planted among the jam to create a living jam. It is will known that both cottonwood and willow sprout readily from buried branches or fragments. Much like the use of biodegradable fabrics to provide short term erosion control until vegetation becomes established to provide long term stability, the incorporation of cottonwood and willow cuttings in the log jam can work to extend the design life considerably.

Figure 12: Large log jam used to provide slope stability along a highway in Oregon. Though not a parallel to jams proposed on the Platte, the coherence of the entire structure is similar and a good example of this stabilizing factor

Permitting Implications

The restoration of the Platte system, and in particular efforts focused on flow consolidation, must occur within the permitting constraints of both federal and state guidelines. Given the dynamic forces at work on the Platte, changes on the scale considered here, from the perspective of ecological impacts, may be relatively short lived and easily reversible. As a testament to this dynamism, an examination of aerial photos dating back to the early 1990's illustrates a much different channel pattern through the CTWR property, and in particular the South channel than what is noted today. Flora and, in particular, faunal assemblages within the Platte have evolved to adapt in this dynamic environment, where niche habitats develop - are occupied - abandoned – and physically fade sometimes within the hydrograph of a single flood event. To induce long term impacts on the Platte system, physically substantial structures must be employed, or, as in the case of water withdrawal and phragmites, a persistent and systemic change within the entire system. The approaches to flow consolidation are neither of these. By utilizing natural materials and mimicking historic habitat forms like log beaver dams (Figure 13) and substantial sand deposits, the project may increase habitat diversity by creating flow refugia during high flow, and slack water pools during low flow periods.

Figure 13: Beavers are still active at CTWR. Approaches to flow consolidation are not far removed from the dams created by beavers

Modeling indicates that changes to the flood regime of the Platte resulting from the project are likely minor. As noted above, the model predicts some increase in flood elevations immediately below CTWR along the left bank where an existing hay field is located, but the impact disappears just below the CTWR property. Flow unconsolidation will occur through flow paths just downstream (0.5 miles) of the CTWR property boundary where the Main channel and the South channel combine. A short distance below this (1.5 miles) all three channels merge at the Elm Creek bridge.

RECOMMENDATIONS

The results of this analysis indicate that flow consolidation can be affected on the Platte River and although challenges are presented with different approaches, the presence of a "fatal flaw" was not uncovered. As mentioned, the Platte is a dynamic environment that does not lend itself easily to quantification through models. Models provide an understanding of trends and potential factors that are not easily observed in the field, but fall short in providing specific conclusions and solid assessments on the potential success of various approaches. Through the modeling effort the following were noted:

- A single hydraulic control is capable of moving water from the South channel to the Main channel to accomplish flow consolidation
- The upstream effects of this solution appear to be negligible, however, downstream, particularly along the left bank of the Main channel, may require added resolution to fully understand
- The location of the overflow channel does not impact the volume of flow conveyed, but does have some impact on flood elevations relative to its location along the property
- The model and the USGS gages throughout the project reach are not in full agreement with respect to predicted and observed results.
- The USGS data indicates that the Platte may be closer to achieving the minimum flows proposed under flow consolidation than the HEC RAS model currently predicts

The usefulness of further reconciling the model to exact conditions on the site can only be determined by the Program. Hydraulic impacts to adjacent landowners are of utmost concern and given that the 8,000 ft^3/s flow may be experienced in 2 of every 3 years pursuing detailed model calibration on these grounds may be warranted. A decision on the level of effort to place on quantitative modeling should be made before considering the physical implementation of flow consolidation.

In addition, it appears that some level of flow consolidation may already be occurring on the CTWR site. When coupled with the parallel efforts of SDHF and Sediment Augmentation, the physical evolution toward flow consolidation that could be in process, may be accelerated by an increase in these two variables. Process based approaches to channel restoration (or flow consolidation) must acknowledge that no one factor can produce the desired result. However, the manipulation of two very prominent fluvial variables, flow and sediment, on the Platte may be enough to achieve the desired outcomes of flow consolidation; a dynamic Main channel with the characteristics of a braided system, without actually utilizing any of the approaches outlined below. Waiting to evaluate such factors as they are implemented must be balanced however with the time sensitive goals of the Program. If flow consolidation must be implemented immediately, an outline for this approach is below.

To implement flow consolidation, we recommend a two phase approach. The first phase is a pilot phase, where the physical implications of blocking and moving flow between the South channel and Main channel can be evaluated at relatively low cost, and permanence, in the landscape. This phase should be accomplished by first digging a pilot channel from the South channel to the Main channel. The location of this overflow channel should be considered with respect to the implications of flooding the adjacent hay field and the advantage of having a "control" reach through the upstream portion of the property. For discussion purposes, assume all flood implications are resolved and the overflow channel is placed in the location consistent with Scenario 5, (see Concept Drawings in Appendix D). Once the overflow channel is excavated, place a major hydraulic control on the South channel in the form of a sand plug, just downstream of the channel entrance. The crest of the plug should be placed at an elevation shown by modeling to achieve the flow values necessary for consolidation. Utilize the results of the HEC Geo RAS model output to construct further minor hydraulic controls on adjacent channels, such that flow will be conveyed toward the Main channel and not to the south of the hydraulic controls on the South channel. Once these temporary sand controls are in place, monitor their performance at the 8,000 ft^3/s event and at lower events to understand water levels in the South channel and the impact that may occur to habitat. If modifications are required with respect to number, spacing, elevations, or size of the plugs, make such modifications and reevaluate under a second flow event. Ultimately observations should be evaluated among three categories or questions:

1. Are changes to habitat within the South channel acceptable or evident?
2. Is the resilience of the structures in the channel acceptable (frequent or infrequent failure)?
3. Is flow consolidation and the desired outcome observed on the Main channel?

Should the pilot phase yield acceptable results and a more permanent approach is desired, especially if more porous hydraulic controls are desired, proceed into the second, more permanent phase by constructing major hydraulic controls using large wood materials harvested from the site. It is prudent to construct a small version of a log control structure, perhaps even during Phase I, to gain site specific insight into their function in the sand bed system of the Platte. Observations gained from this prototype can be applied to the larger structures. To ensure the longevity of these features by planting live stakes of cottonwood and willow, and cable all logs together as noted in the Concept Plans (Appendix D) such that the structures are cohesive and persistent in the South channel. Continue to evaluate following floods in excess of the SDHF events, as well as at low flows to ensure that desired outcomes are being achieved. Utilize adaptive management to maintain desired effects, or induce more permanent controls.

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APPENDICES

A	–	Hydraulic Modeling Detailed Report
B	–	HEC Geo RAS Output Maps
C	-	Unsteady vs. Steady Modeling Analysis Report
D	-	Concept Drawings

APPENDIX A – Hydraulic Modeling Detailed Report

The goal of flow consolidation is ensuring that 85% of the 8,000 ft^3/s flow event is conveyed through the Main channel. An available HEC Geo-RAS hydraulic model of the Platte River system within and near the Cottonwood Ranch property was a primary tool in examining opportunities for meeting this objective. The results of this effort are documented below.

HEC – GEO RAS Model

The HEC-RAS model provided is a detailed model, developed and calibrated in 2009 for the Program by Tetra Tech under contract with HDR. For the purposes of this study, the section of the model downstream of the Overton Bridge and upstream of the US 183 (Elm Creek) bridge was the main focus. Within this section, the system features a North channel, Main channel, South channel, and several secondary connections between these channels. The model includes 22 hydraulic sections through the Cottonwood Ranch (CTWR) property, between stations 101253 and 122498, with distances between them ranging from approximately 500 ft to 2000 ft. The model included a broad range of flows. For this study we primarily focused on flows that have occurred during the period of record for the available USGS gage stations and the 8000 ft^3/s target flow. The Manning's roughness values used in the model were typically on the order of 0.03 for the channel, which is appropriate for clean, sandy streams with limited vegetation within the stream, and 0.1 for the floodplain, which is appropriate for floodplains with dense trees and brush. The model identified extensive areas of ineffective flow, or areas where water would be expected to pool, but not actively flow. Some of these areas coincide with side channels, expected to convey water during high flows so we tested the potential error associated with mis-identifying these areas as ineffective flow by changing them to effective flow areas in the model. The model predicted only minor changes in flow quantity, due to the relatively small size of these side channels.

RELEVANCE OF THE AVAILABLE MODEL

Discussions with Tetra Tech indicated that the model was created with a variety of survey information and was calibrated to the extent possible using gage data, aerial photos at known flows, water surface surveys, and other information. Given the extent of the model (Lexington, NE to Chapman, NE), the various data sources and calibration methods, the model results were deemed by both Program staff and

Tetra Tech to be sufficient for evaluating macro-scale changes on the CTWR site with respect to flow consolidation. This assumption was appropriate for the level of analysis being performed at this feasibility stage. However, evidence provided by cursory investigations into the predicted vs. observed model output indicate that under a Final Design scenario, additional effort may be required to reconcile modeled and observed results if a high level of hydraulic resolution is required.

June 2010 Event

There are three USGS gage stations currently maintained within and just upstream of the Cottonwood Ranch property. One gage is located upstream of the Overton Bridge (USGS Gage #06768000) and represents the entire river flow consolidated within one channel. The second gage is on the South channel (USGS Gage #06768025) approximately 2.7 miles downstream from the bridge. The third gage is on the Main channel (USGS Gage #06768035) approximately 5.3 miles downstream from the bridge (Figure 1). The gage on the South channel in particular collects discharge data specific to that channel. It does not capture flow that may spill out of the South channel to either the north or south of the gage.

Figure 1: USGS gages near the Cottonwood Ranch property

A high flow event in June 2010 provided data to compare the HEC RAS model results with recorded stage and discharge at these three gages (Figure 2). Gage heights were not consistently available during this flood period at all three gages. However, on June 27, it appeared that all three gages provide a realistic snapshot of measured stage and discharge with which to compare to HEC RAS model results. The June 27 flow at Overton was slightly less than the flow discharge already in the HEC RAS model but still yielded useful information (7,220 ft^3/s vs. 7,370 ft^3/s). This flow was modeled, and results were compared to USGS reported flows and water surface elevations for that day (Table 1).

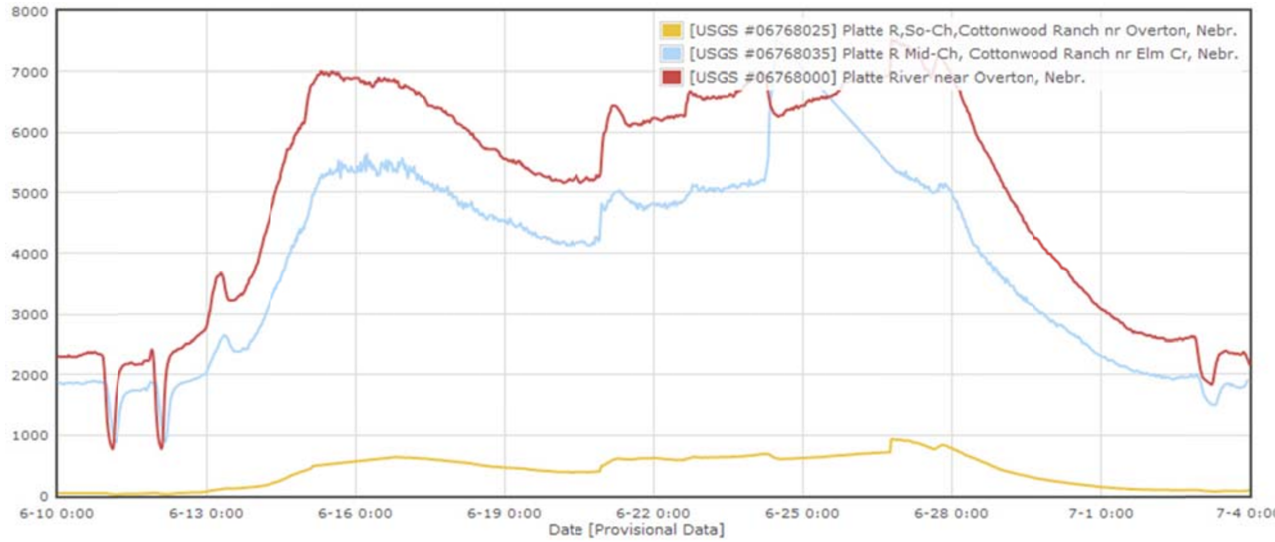

Figure 2: USGS Hydrograph of June 2010 flood event.

	Overton Bridge Gage (Main 6, Section 136448 in model)		**South Channel Gage** (Split H, Section 32613 in model)		**Main Channel Gage** (Main 11, between Sections 108163 and 107688 in model)	
	Flow (ft^3/s)	**WSE (ft)**	**Flow (ft^3/s)**	**WSE (ft)**	**Flow (ft^3/s)**	**WSE (ft)**
Modeled (HEC RAS)	7370	2303.35	1528	2285.28	5065	2269.60
Observed (6/26/10)	7370	-	877	-	5740	-
Observed (6/27/10)	7220	2304.31	872	2284.84	5160	2267.80

Table 1: Model vs. Observed (USGS gage) results on June 27, 2010 at Cottonwood Ranch. June 26 WSE data were not available, but discharge estimates coincide exactly with model output and were included for that purpose

The model predicted flow in the Main channel approximately 700 ft^3/s lower than that observed, while it predicted flow in the South channel that was higher than that observed by approximately the same amount. The modeled water surface elevation at the Overton Bridge was almost 1 ft lower than that recorded at a slightly lower flow. Further downstream, the modeled Main channel water surface elevation was almost two feet higher than the recorded elevation at a slightly higher flow. This suggests that the model may be predicting a flatter hydraulic slope through this reach than currently occurs at this flow. The peak flow water surface elevation predicted by the model in the South channel is less than half a foot higher than the recorded elevation when the actual flow is only 57% of the modeled flow.

In addition to the gage data for the June event, an infrared aerial photograph of the CTWR property was taken on 6/17/2010 during this same high flow period. Flow on the day of the photograph at the Overton Gage was recorded near 7,000 ft^3/s and was steady throughout the day. Comparing the horizontal extent of water distribution throughout the site on the aerial with the modeled results as a Geo RAS output allowed a coarse level comparison of predicted water distribution on the site. This comparison illustrated insignificant differences between the extent of water in the aerial photo and that predicted in the Geo RAS model. It is important to note that from a planview perspective, until channels of the Platte exceed their bankfull capacity (well in excess of an 8,000 ft^3/s event) the Geo RAS results illustrate only minor differences, as the plan view does not capture vertical changes effectively. These results at 7,000 ft^3/s and a range of flows, overlaying the 6/17/10 photo are shown in Appendix B.

APRIL 7, 2011 SITE VISIT

A second opportunity to assess the HEC-RAS model occurred on 4/7/11 during a site visit with the project partners. Flow was steady on 4/7/11 throughout the day (Figure 3) at about 3,700 ft^3/s. Three cross sections were taken with an auto level to quantify the water surface elevation of flow in the Main channel compared to the South channel (Figure 4).

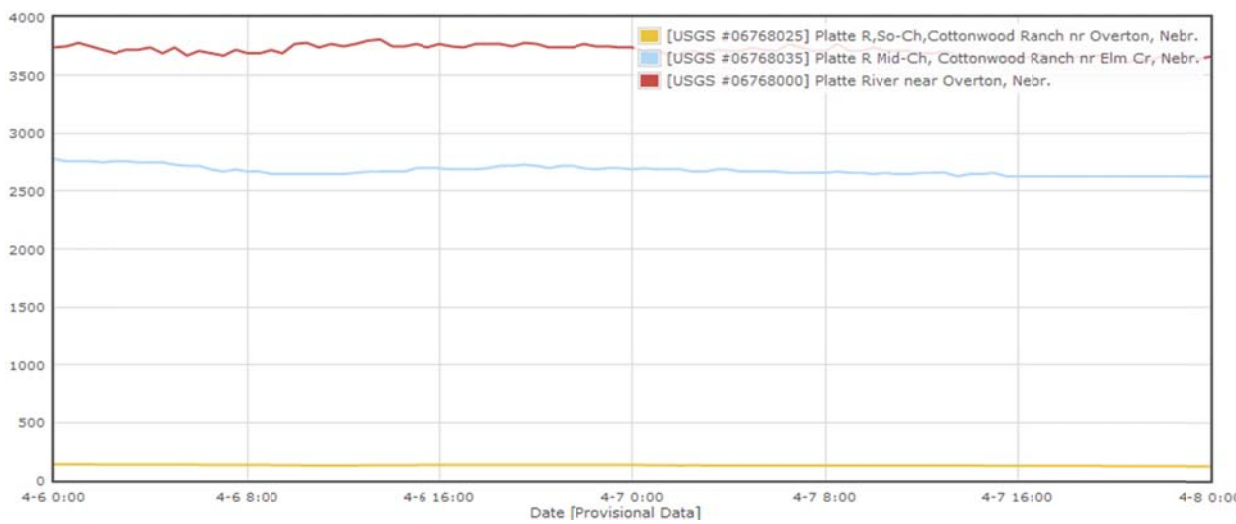

Figure 3: USGS observed flow 4/6-4/8 2011

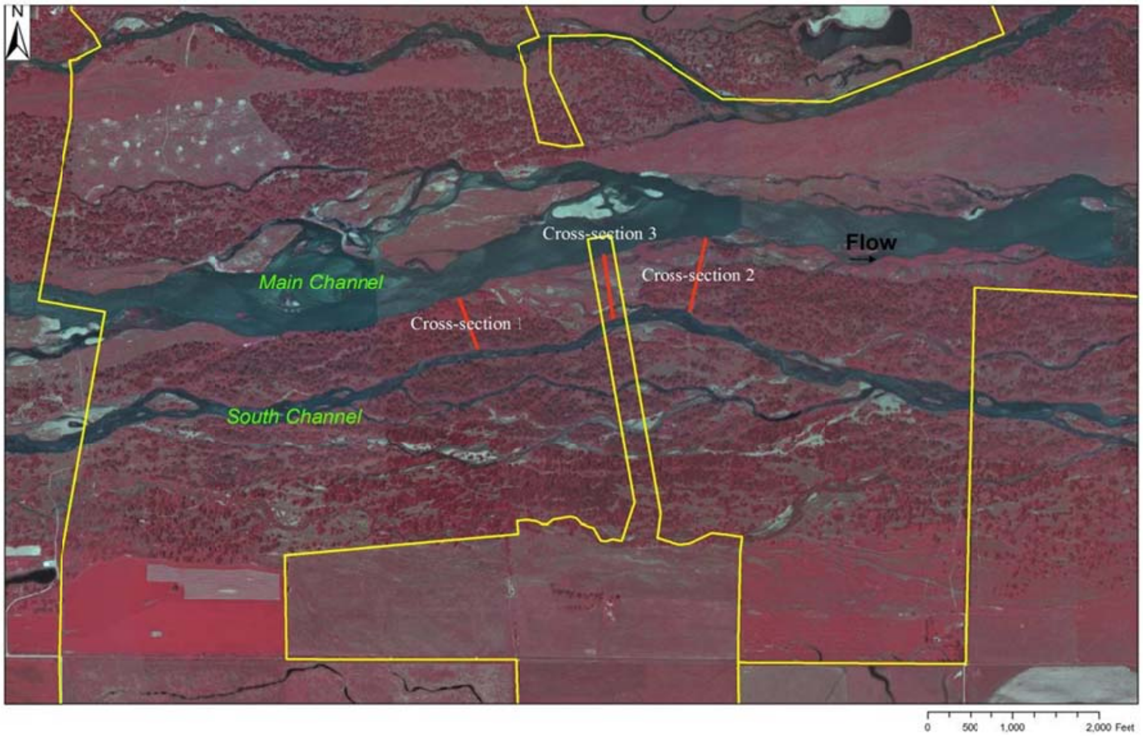

Figure 4: Cross section locations comparing water surface in Main channel and South channel, 4/7/11

Survey data collected during this site visit suggested that the head differential between the Main channel and the South channel ranged from 2.47 ft to 3.28 ft (Table 2). Running the existing conditions model with a flow of 3710 ft^3/s, recorded by the Overton gage (USGS #06768000) allowed a comparison of predicted head differences to those observed on site (Table 3). These differences indicate that water distribution among the channels at CTWR predicted by the HEC RAS model are different than those observed.

Survey Section	Main Channel Station (HEC model)	Difference in WSE (Main – South =)	
		Measured	Modeled
1	118+498	3.11’	1.67’
2	115+321	2.47’	2.11’
3	116+137	3.28’	1.59’

Table 2: Surveyed and Modeled comparison of difference in water elevation Main channel and South channel

	Overton Bridge (Main 6, Section 136448 in model)		South Channel (Split H, Section 32613 in model)		Main Channel (Main 11, between Sections 108163 and 107688 in model)	
	Flow (ft^3/s)	WSE (ft)	Flow (ft^3/s)	WSE (ft)	Flow (ft^3/s)	WSE (ft)
Modeled (HEC RAS)	3710	2301.66	366	2283.37	3005	2268.50
Observed (4/7/11)	3710	2300.47	130	2281.9	2670	2266.42

Table 3: Predicted vs. Observed water surface elevations on 4/7/2011

The USGS recorded flows and predicted WSE (water surface elevation) for both the Main channel and the South channel are lower than predicted by the model. In the case of the South channel, it is notable that on 4/18/09, when the actual flow (363 ft^3/s) was very close to that predicted by the model, the water surface elevation was 2283.5 ft, which is very close to that modeled. Therefore, the difference in the recorded and modeled elevations for the flow on 4/7/11 is likely due to the difference in flow rates or a geomorphic change that occurred between April 2009 and April 2011.

Summary of Model Relevance

		Model Output (Hec RAS)		Observed Output (USGS Gages)	
		Flow (ft^3/s)	% of Total	Flow (ft^3/s)	% of Total
7,370 ft^3/s 6/26/2010	South Channel	1528	21%	877	12%
	Main Channel	5065	69%	5740	78%
3,710 ft^3/s 4/7/20101	South Channel	366	10%	130	3.5%
	Main Channel	3005	81%	2670	72%

Table 4: Summary of observed and predicted flow distribution on the Platte for 2 dates. NOTE the balance of flow not accounted for is likely in the ungaged North channel.

There are several factors that likely contribute to the differences between observed and modeled results. The modeled results are based on an assumption that the flow rate continues at the same level for a long period of time, while the actual flow rates were rising and falling at different times and rates in the different reaches. Secondly, this stretch of river is known to receive and contribute to groundwater at different times, which is not reflected in the model. For example, on 4/7/11, both the South channel and the Main channel have significantly lower flows than that predicted by the model. This suggests that the stream may have been losing surface water to groundwater that day, particularly given that it was raining. Finally, some of the differences are likely due to changes that have occurred in the system since the time that the model was developed and calibrated. These changes may have consolidated flow in the Main channel more than the model predicts, which suggests that potentially less additional water needs to be conveyed to the Main channel to achieve the 85% consolidation goal.

Although there are some discrepancies between the model predictions and the recent recorded flow rates and water surface elevations, the model remains a useful tool for examining the feasibility of achieving flow consolidation. The Final Design phase of flow consolidation, if pursued, will require closer scrutiny of these model inconsistencies, largely within the context of off-property upstream and downstream effects to ensure the Program's good neighbor policy is fulfilled. If the latter can be effectively ruled out, then the resources required to calibrate the model vs. performing field tests of prototypes to inform design should be considered. Given the dynamics of the Platte, a calibrated model at the CTWR scale may be valid only until the next significant flood.

UNSTEADY VS. STEADY STATE HYDRAULICS

Inter-Fluve investigated the effect of the varying nature of typical flow events by running the model in an unsteady state mode. This would allow routing dynamic hydrographs through the system rather than assuming constant, steady flow rates. However, comparison of hydrographs at different river stations suggested that very little flow attenuation occurs in this system, and flood peaks do not rise or fall so fast that a steady-state model is not applicable. In addition, the unsteady state model did not include all channels that occur in the system and that are represented in the steady state model. Our understanding is that some of these reaches needed to be deleted to achieve stability in the unsteady model. Deleted reaches in the unsteady model introduced additional flow into the newly designed reach, misrepresenting actual flow processes. As a result, the steady-state model was used for the analysis of flow consolidation scenarios. A brief summary of the unsteady analysis is included in Appendix C.

SCENARIOS TESTED WITH THE MODEL

The goal of this flow consolidation analysis is to determine methods of consolidating at least 85% of the total river flow into the Main channel when the total river flow is 8000 ft^3/s. Therefore, the target minimum flow in the Main channel is 6800 ft^3/s. When the existing conditions are analyzed at this flow level, 5580 - 5450 ft^3/s are predicted by the model to flow in the Main channel through the Cottonwood Ranch property, 750 ft^3/s are predicted to flow in the north channel, and 1670 – 1800 ft^3/s are predicted in the South channel, as shown in Table 4 below. To achieve the target flow in the Main channel, 1220 – 1350 ft^3/s needs to be redirected from the South channel to the Main channel and maintained there through the project reach.

MAIN CH STA	North Q (ft^3/s)	Main Q (ft^3/s)	South Q (ft^3/s)	Q TOTAL (ft^3/s)
122498.2	750	5580	1670	8000
121965.7	750	5580	1670	8000
121049.4	750	5580	1670	8000
120487.5	750	5580	1670	8000
119995.7	750	5580	1670	8000
118497.9	750	5580	1670	8000
117803.4	750	5450	1800	8000
116937.2	750	5450	1800	8000
116137.4	750	5450	1800	8000
115321.2	750	5450	1800	8000
113902.6	750	5450	1800	8000
112981.1	750	5450	1800	8000
112347.1	750	5450	1800	8000
110327.2	750	5450	1800	8000
108689.2	750	5450	1800	8000
108163.8	750	5450	1800	8000
107688.9	750	5450	1800	8000
106097.9	750	5450	1800	8000
104021.8	750	5450	1800	8000
103291.6	750	5450	1800	8000
102157.5	750	5450	1800	8000
101253.4	750	4010	3240	8000

Table 5: Existing flow values at 8,000 ft^3/s predicted in HEC RAS

Scenario 1 - New Overflow Channel

To facilitate the movement of water from the South to the Main channel, an overflow channel was necessary. A reasonable location near the upstream end of the property was identified. A new reach was added to the model in this location (see Figure 5). As discussed above, it was estimated that the new overflow channel needs to carry at least 1220 ft^3/s from the South channel to the Main channel in order to satisfy design criteria. Using Manning's equation, it was determined that a channel with a bottom width of ~80 ft side slopes of 2:1 and depth of 5-6 feet would be necessary to convey at least 1220 ft^3/s given the slope. A channel approximately 1720' in length, with these dimensions was added to the model.

At 8000 ft^3/s, the energy grade elevation in the South channel at the entrance to this new channel is 2286.23 and at the outlet in the Main channel it is 2285.41. Therefore, the energy grade slope in the new channel would be 0.0005. The energy grade slope in the South channel downstream of this point is 0.003. Thus, the energy slopes strongly favor flow through the South channel rather than in the new overflow channel. When the model was run with this new channel but no other changes, only 150 ft^3/s flowed through the overflow channel to the Main channel, far short of the 1220 ft^3/s target. It was determined that changes to the South channel would be necessary to increase the head in the South channel and force additional water north to the Main channel. Results are summarized in Table 6 and Figure 6 below.

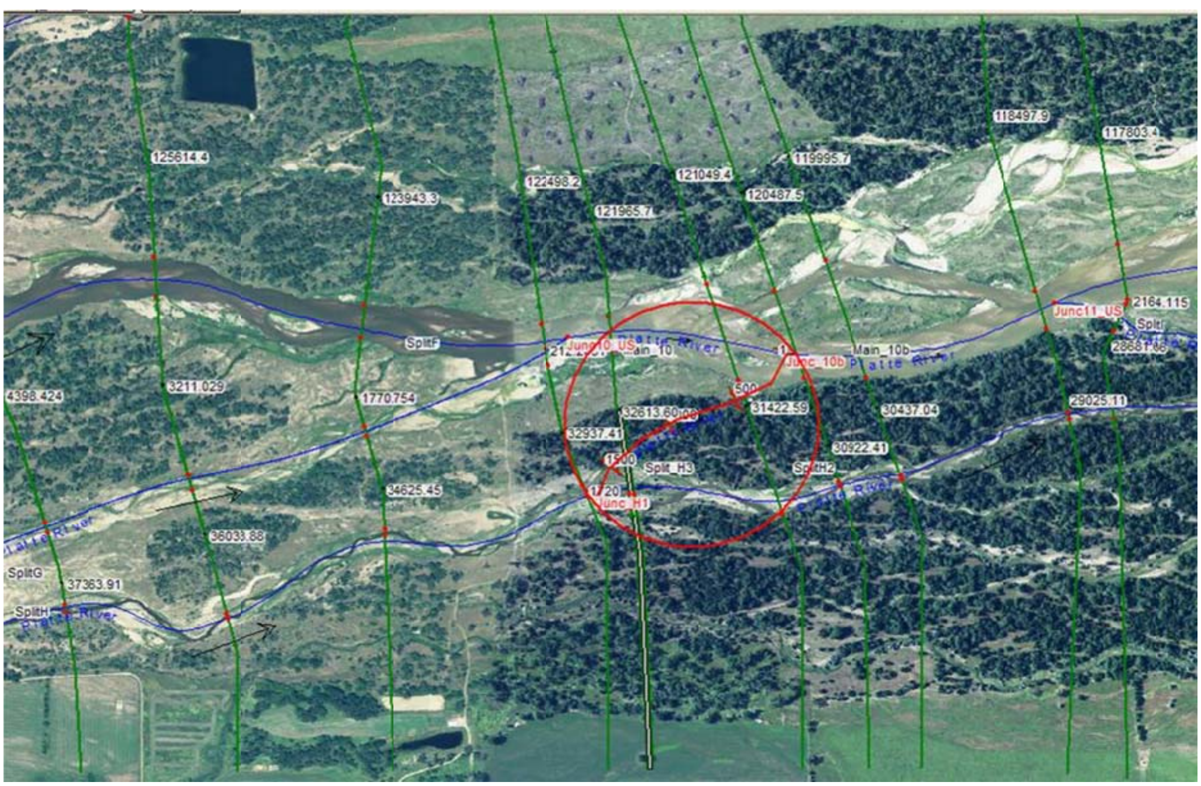

Figure 5: Location of new overflow channel on the CTWR sight in the HEC RAS model

	Existing	Scenario 1 – New channel
South Ch. Discharge (ft³/s)	1670	1520
Overflow Ch. Discharge (ft³/s)	0	150

Table 6: Flow results of Scenario 1

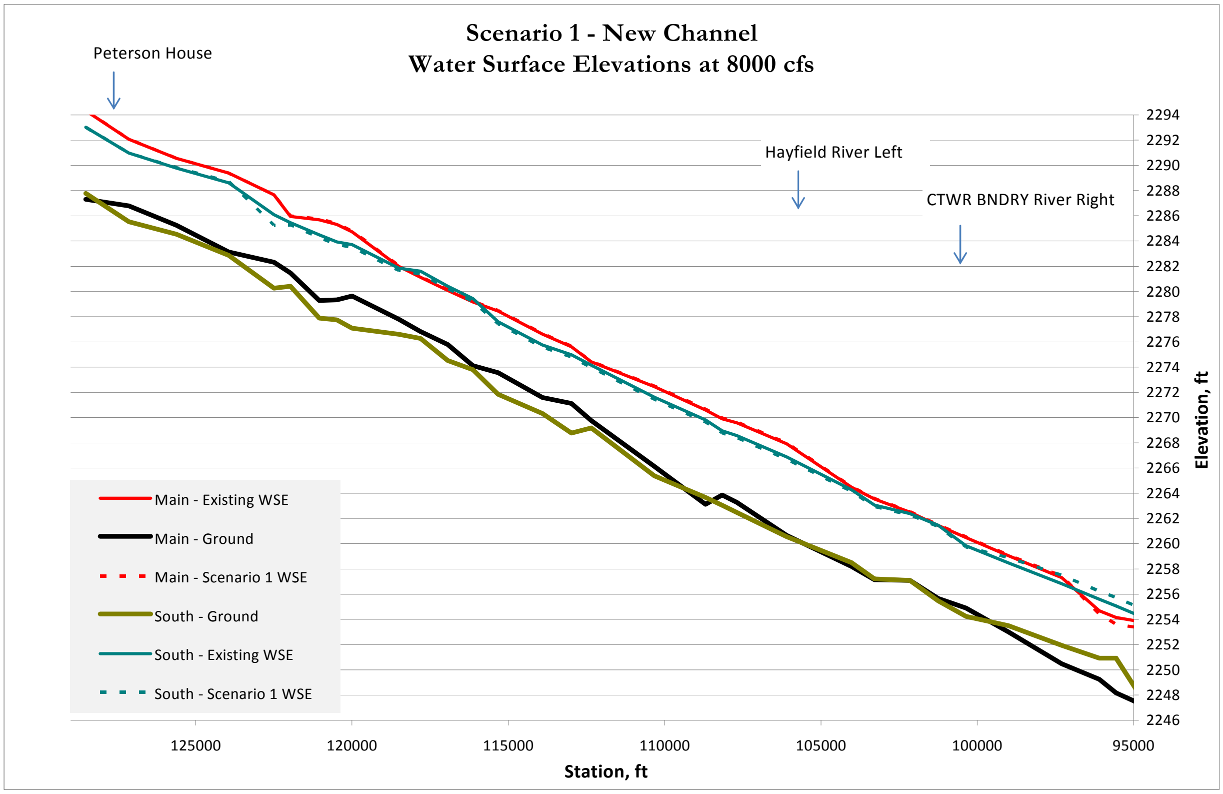

Figure 6: Predicted WSEs (water surface elevations) from the HEC RAS model

Scenario 2 – Overflow Channel + Increased Channel Roughness

Increasing the channel roughness in the South channel may slow water sufficiently to increase head, forcing more water through the overflow channel into the Main channel. We simulated this in the model by replacing all Manning's n values less than 0.1 to 0.1 in the reaches of the South channel within the property. The increase in roughness boosted the WSE in the South channel just downstream of the new channel from 2285.4 ft to 2286.0 ft and increased the flow in the new channel from 150 ft³/s to 880 ft³/s (Table 6). This caused an attendant increase in Main channel discharge to 6,460 ft³/s, or 81% of the total flow. Barnes (1967) summarized computed Manning's n for a variety of channel types and the largest n values he found were on the order of 0.075-0.079. Therefore, even if roughness were maximized beyond perhaps practical limits, the goal of achieving 85% of the flow in the Main channel would not be attained solely by roughening the South channel. Results are summarized in Table 7 and Figure 7 below.

	Existing	Scenario 1 - Overflow channel	Scenario 2 – Overflow channel + Added roughness
South Ch. Discharge (ft³/s)	1670	1520	790
Overflow Ch. Discharge (ft³/s)	0	150	880

Table 7: Flow results of Scenario 2

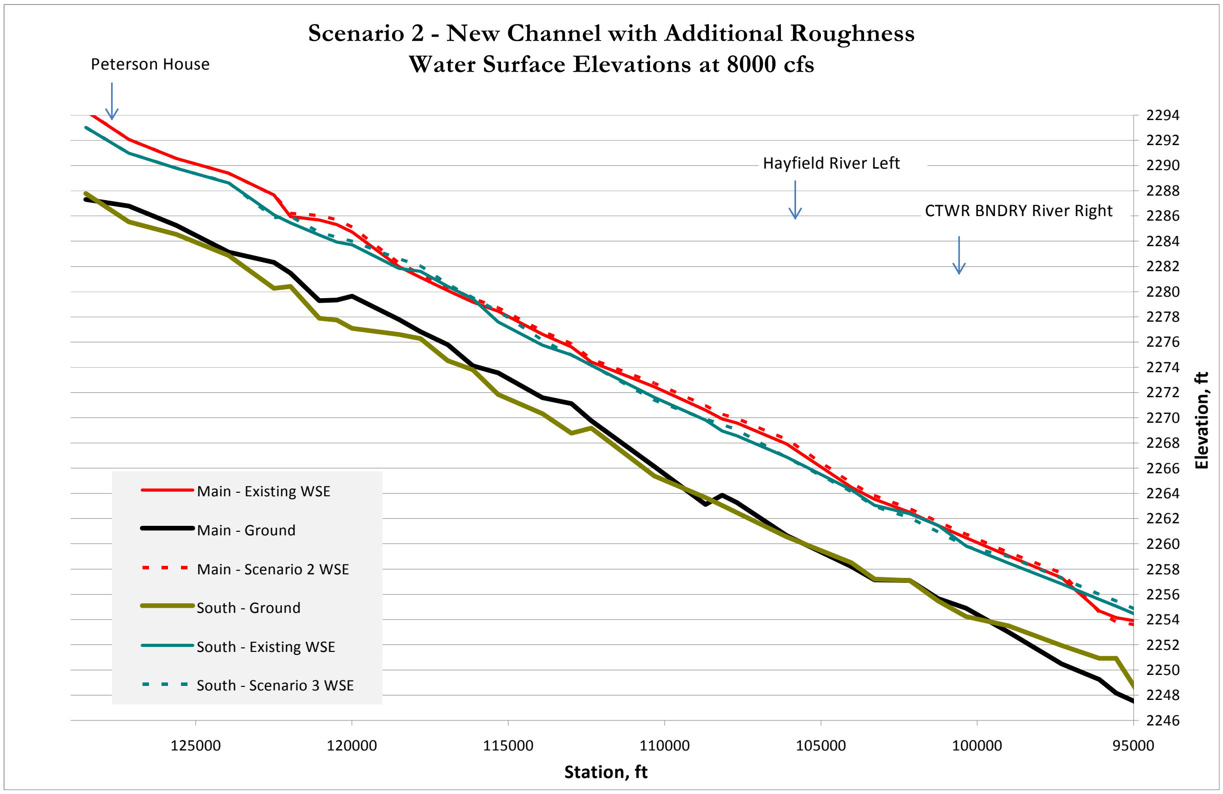

Figure 7: Predicted WSEs (water surface elevations) from the HEC RAS model

Scenario 3 – Overflow Channel + Single Hydraulic Control

Without significantly increasing the hydraulic head in the South channel, the hydraulic gradient in the South channel continues to favor flow in this channel instead of the overflow channel. The roughness values were returned to the calibrated values reflected in the original model, and an inline structure (hydraulic weir) was added to the model on the South channel downstream of the new split to simulate a partial blockage and force water into the new channel. If constructed, this control could be inserted in the form of an engineered log jam on the site. Consistent with the porous nature of such a structure, we adjusted the opening in the weir to simulate this condition. The opening in the structure had to be restricted to just 52.5 sq ft (15 ft wide by 3.5 ft high) to force enough water through the new channel. The elevation of the crest of this structure was 2287', about 0.5' below the top of bank, or perhaps the top of the terrace at that location on the South channel. Running the model with this scenario resulted in a head across the structure of 3.3 ft., though the increase in water surface elevation at this cross section was only about 1' above existing conditions. The flow through the new channel is 1375 ft^3/s, creating a flow in the Main channel of 6957 ft^3/s which is 87% of 8000 ft^3/s . This scenario satisfies the minimum flow consolidation value. Results are summarized in Table 8 and Figure 9 below.

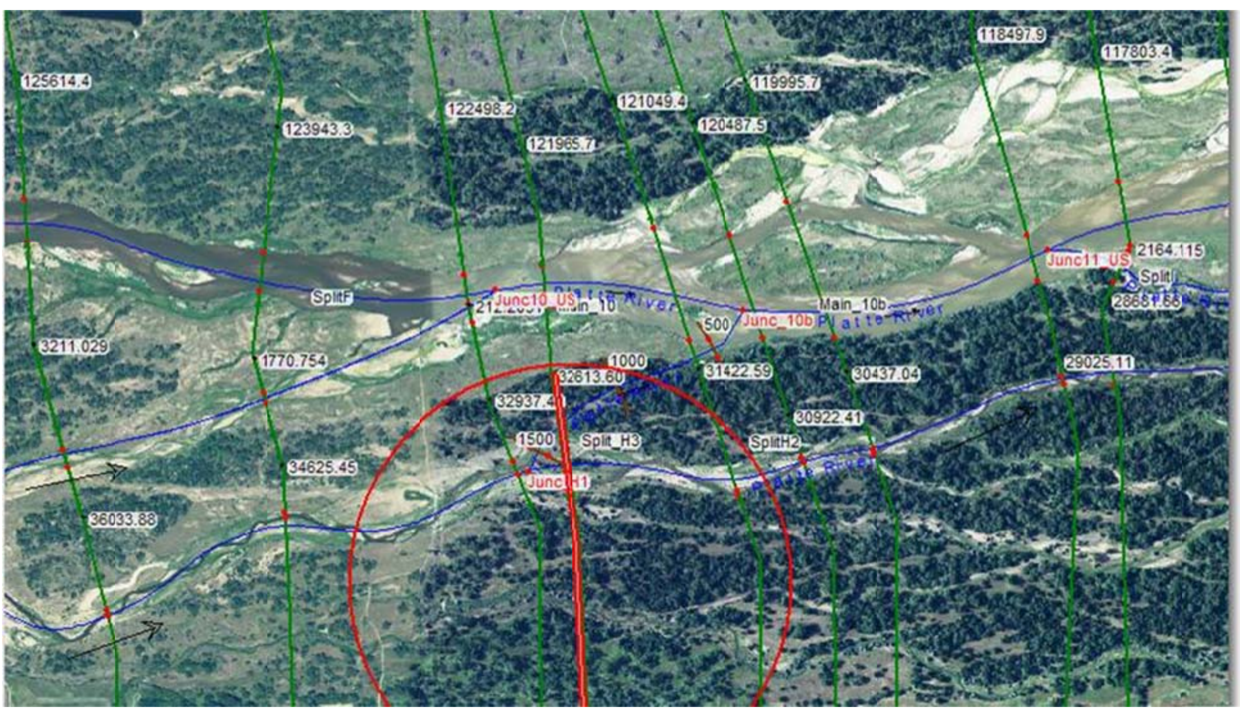

Figure 8: Location of hydraulic control (blockage) placed below the overflow channel in the HEC RAS model

	Existing	Scenario 1 - Overflow channel	Scenario 2 – Overflow channel + Added roughness	Scenario 3– Overflow channel + Hydraulic Control
South Ch. Discharge (ft^3/s)	1670	1520	790	330
Overflow Ch. Discharge (ft^3/s)	0	150	880	1340

Table 8: Scenario 3 flow results, the highlighted scenario meets design criteria

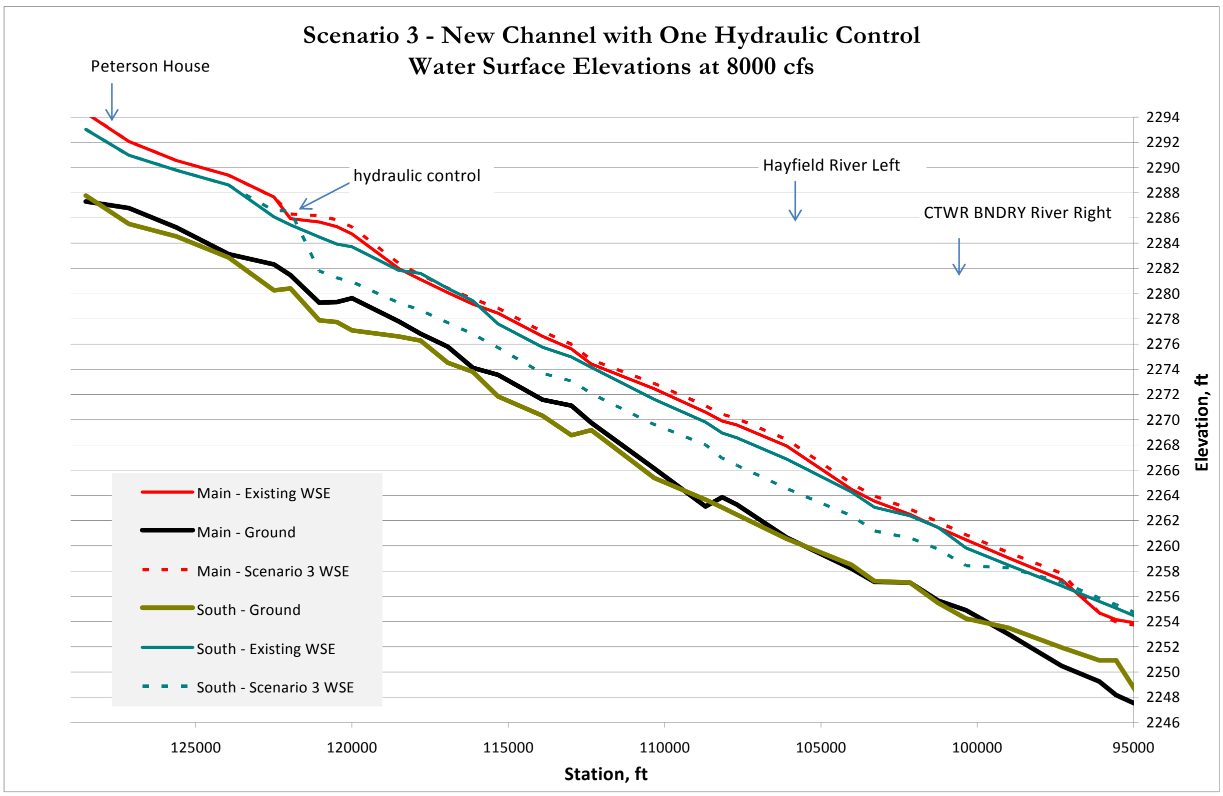

Figure 9: Predicted WSEs (water surface elevations) from the HEC RAS model

Scenario 4 – Multiple Hydraulic Controls

If no other modifications to the system are made, a portion of this new flow into the Main channel is predicted in the model to return to the South channel through the reach (Split I in the model) that cuts back to the South channel further downstream.

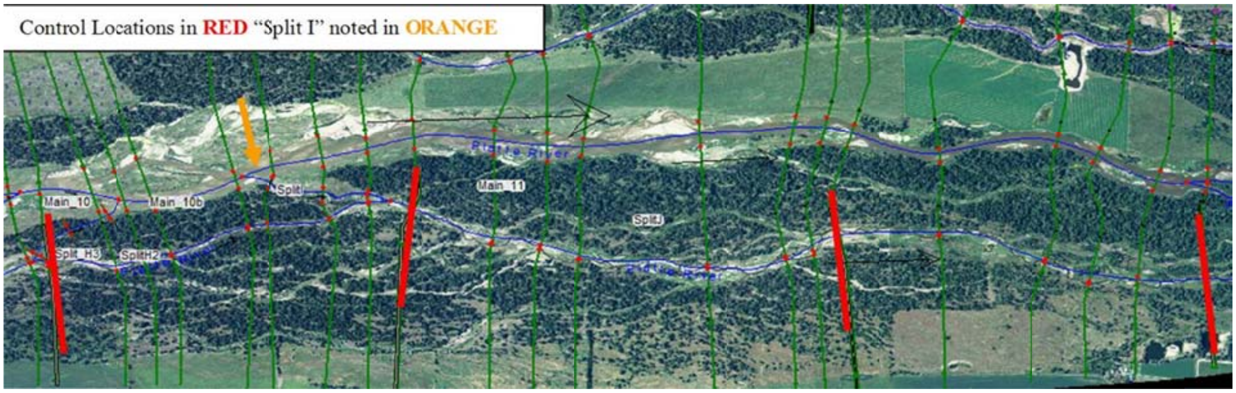

Figure 10: Plan view of Split I connecting the Main channel back to the South channel below the overflow channel

Additionally, the decreased flow in the South channel results in a lower water surface elevation that may encourage flow through the sandy soils from the Main channel to the South channel. A scenario was developed in HEC RAS that included a blockage at Split I to minimize water through that channel and also included multiple blockages along the South channel to increase the water surface elevation to reduce the groundwater gradient along that channel. The model results (Table 9) indicate that this too will achieve the flow criteria necessary. The water surface profiles for the Main channel and South channel are shown in Figure 11.

	Existing	**Scenario 1 - Overflow channel**	**Scenario 2 – Overflow channel + Added roughness**	**Scenario 3– Overflow channel + Hydraulic Control**	**Scenario 4– Overflow channel + Multiple Hydraulic Controls**
South Ch. Discharge (ft^3/s)	1670	1520	790	330	330
Overflow Ch. Discharge (ft^3/s)	0	150	880	1340	1340

Table 9: Scenario 4 flow results, the highlighted scenarios meet design criteria

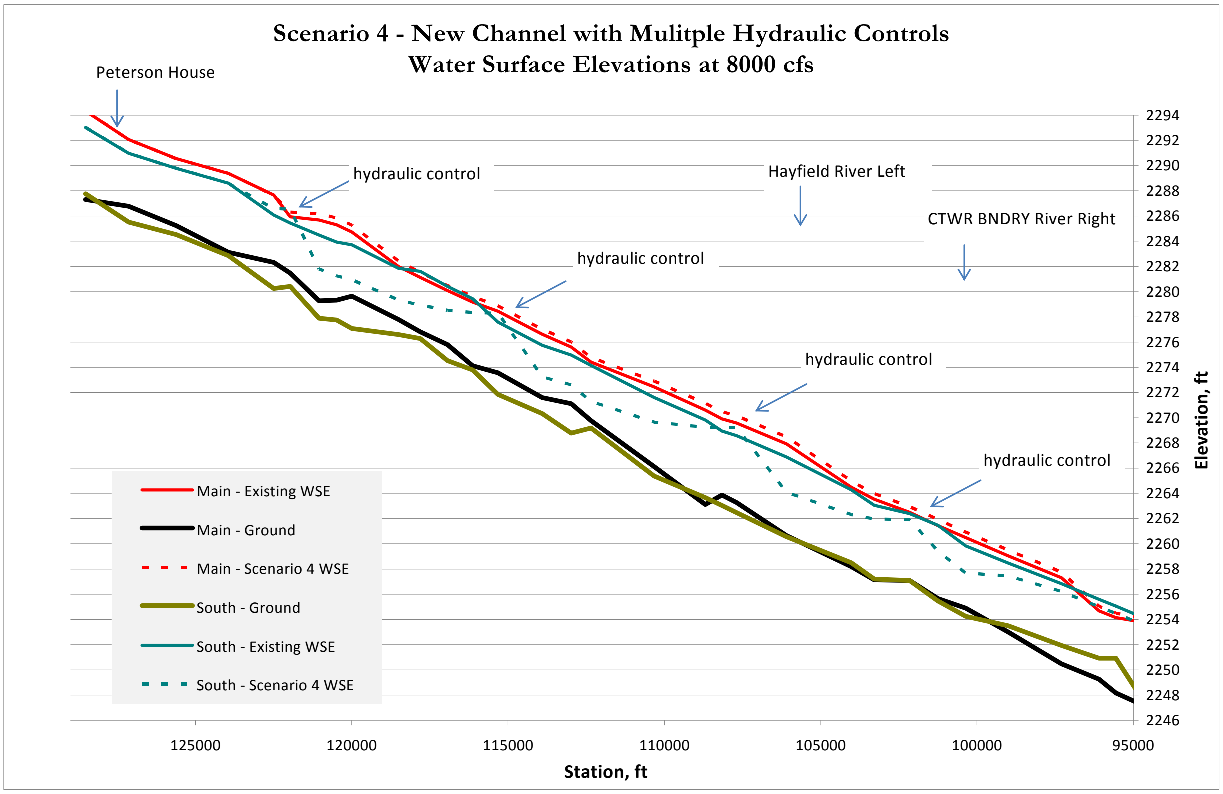

Figure 11: Predicted WSEs (water surface elevations) from the HEC RAS model

Scenario 5 - Alternate location for the new channel

Discussion with project partners indicated a desire to investigate the possibility of moving the overflow channel to a location further downstream than had been modeled in Scenarios 1-4. The benefits of moving the channel downstream include: (1) reduced risk of flooding an access road just upstream of the original channel location, (2) allow a "control" section of the Main channel within the property boundary to study the effect of flow consolidation, and (3) move the partial blockage to a portion of the channel that is more constricted, requiring less material to effectively achieve the block.

Given these anticipated benefits, we modeled a scenario that located the overflow channel further downstream (see Figure 12). As with Scenario 3, a partial blockage was simulated on the South channel downstream of the new channel to increase the head upstream to force more than 1220 ft^3/s through the new channel. The head across the blockage to achieve this flow is 3 ft. However, as discussed above, it may not be necessary to push 1220 ft^3/s across to the Main channel, because a portion of this flow may currently be in the Main channel already. Results of this analysis are in Table 10 and Figure 13 below.

Figure 12: Location of new overflow channel, further downstream on the CTWR property

	Existing	Scenario 1- Overflow channel	Scenario 2– Overflow channel + Added roughness	Scenario 3– Overflow channel + Hydraulic Control	Scenario 4– Overflow channel + Multiple Hydraulic Controls	Scenario 5– *Downstream* overflow channel + Hydraulic Control
South Ch. Discharge (ft^3/s)	1670	1520	790	330	330	370
Overflow Ch. Discharge (ft^3/s)	0	150	880	1340	1340	1300**

Table 10: Scenario 5 flow results, the highlighted scenarios meets design criteria
**As discussed further below, this scenario may result in unwanted increases to flood elevations

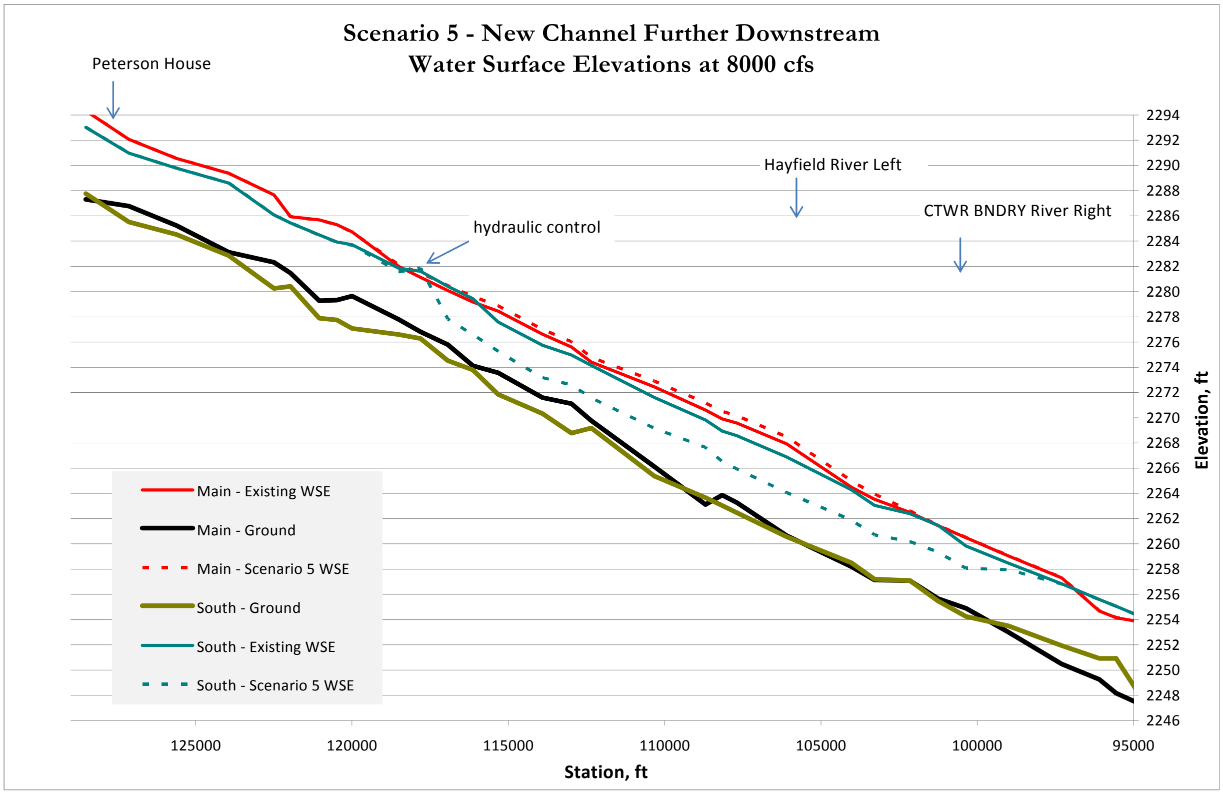

Figure 13: Predicted WSEs (water surface elevations) from the HEC RAS model

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OFF PROPERTY HYDRAULIC CHANGES

To investigate whether proposed changes to the river system would affect flooding upstream or downstream of the CTWR property, water surface elevations at several locations were reviewed in more detail. Table 11 below shows the water surfaces in the Main and South channels at the upstream end of the property and downstream end of the project (station 99004) for both the existing conditions and for Scenario 5 at 8,000 ft^3/s total flow. Water surface elevations at these locations are predicted to be identical under both existing conditions and scenario 5 (the preferred scenario) at the upstream end of the property. However, at the downstream end of the property, the South channel water surface is considerably lower because water that was routed to the Main channel upstream has not had an opportunity to return. Downstream of the CTWR property, there is a channel that carries water from the Main channel back to the South channel. The first cross section downstream of this channel is ~4000 feet downstream of the property. At this point, the water surface elevations are predicted to be identical for the existing conditions and Scenario 5 in both the Main and South channels.

	Model Reach and Station	Existing Conditions WSE (ft)	Scenario 5 WSE (ft)
South Ch. Upstream	Split H Sta 34625	2288.6	2288.6
Main Ch. Upstream	Main 9 Sta 123943	2289.4	2289.4
South Channel near Downstream End of Prop	Split J or H2 Sta 13033	2262.4	2260.2
Main Channel near Downstream End of Prop	Main 11 Sta 102157	2262.5	2262.6
South Channel ~ 4000 ft downstream of property	Split L Sta 7633	2256.8	2256.8
Main Channel ~ 4000 ft Downstream of Property	Main 12 Sta 97304	2257.3	2257.3

Table 11: Existing and Proposed WSE comparison as predicted by the HEC RAS model

Another location where changes to water surface elevations were reviewed closely is near the hay field just north of the Main channel near the downstream end of the property. The Main channel cross sections that reflect conditions in this area are at stations 106098, 104022, and 103292. Plan view location and cross sections from the HEC RAS model are shown in the figures below.

Figure 14: Plan view of HEC model cross section. The hayfield can be seen in upper center of the Figure. Data for XS 106097, 104201, and 103291 are shown below

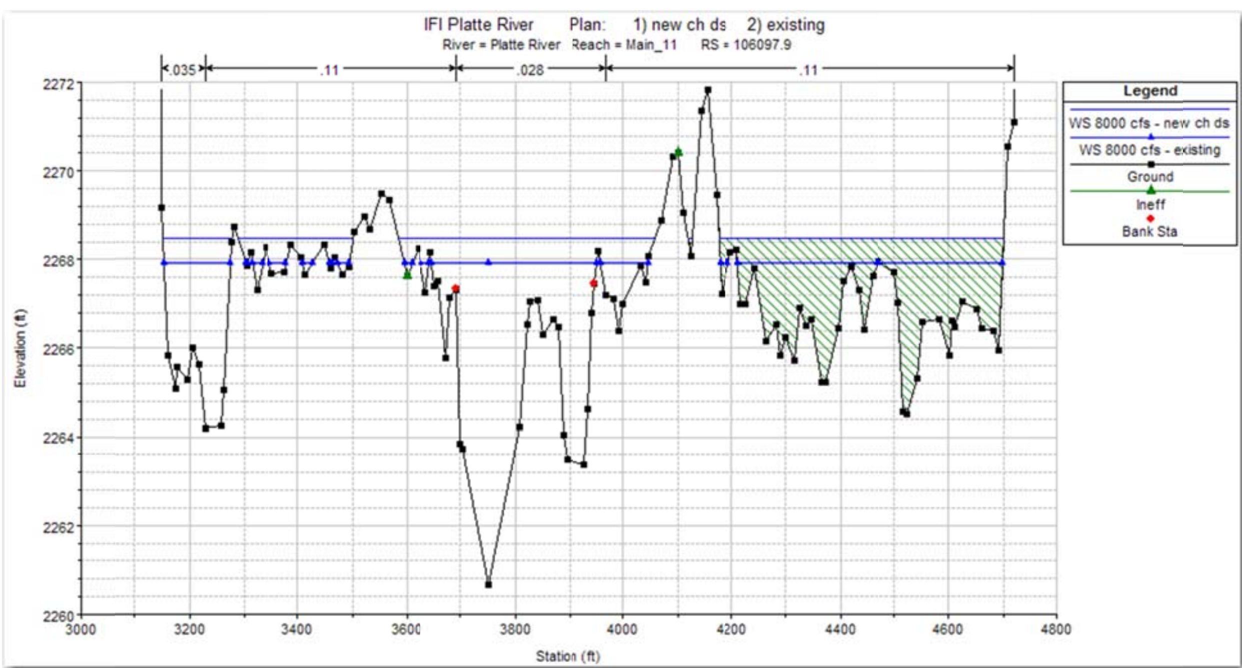


Figure 15: HEC results for XS 106097 showing a slight rise in WSE in the hayfield on river left

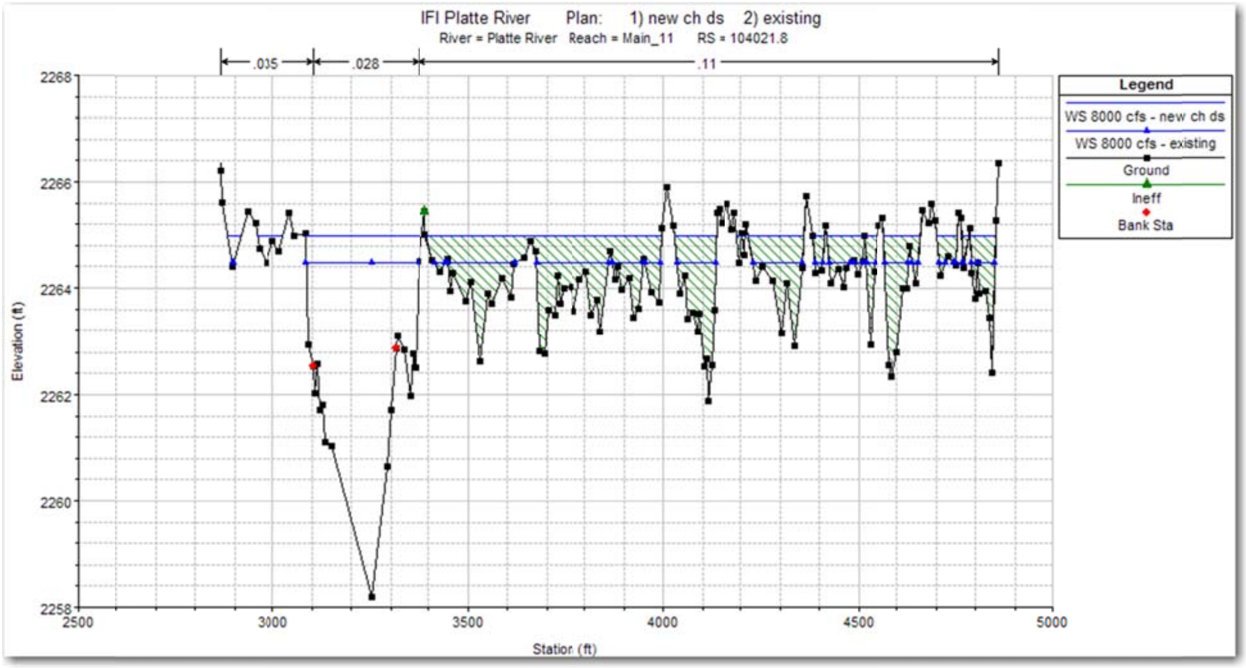


Figure 16: HEC results for XS 104021 showing a slight rise in WSE in the hayfield on river left

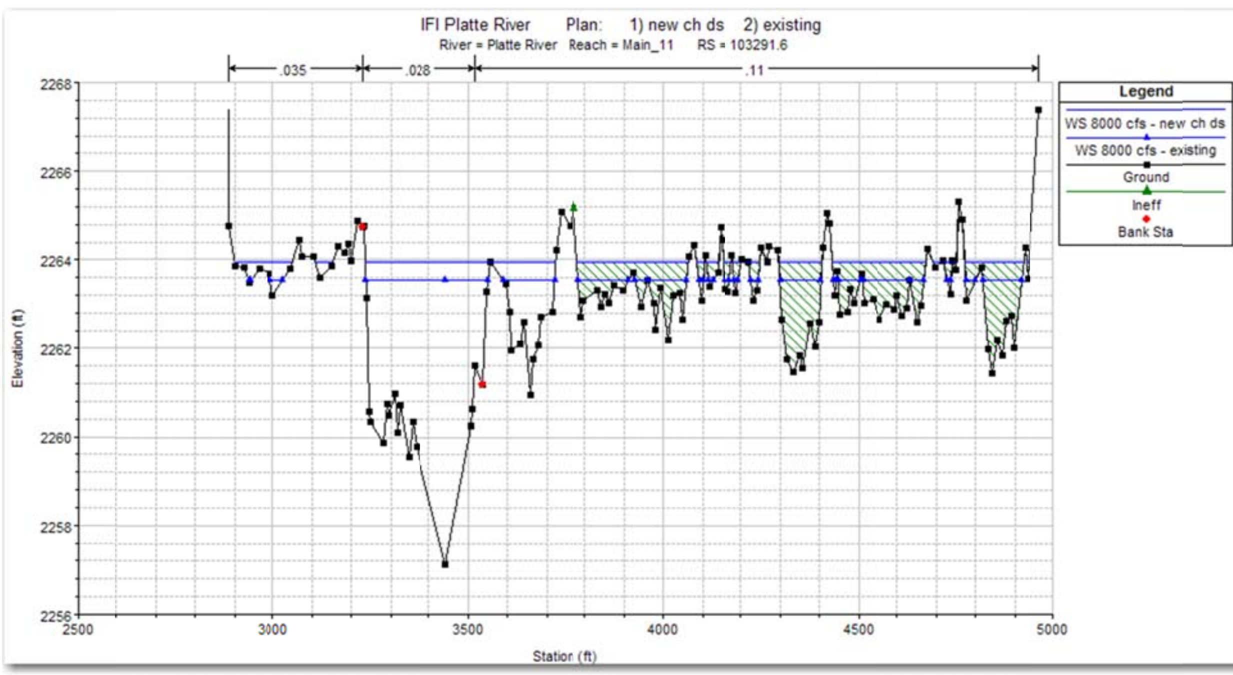


Figure 17: HEC results for XS 103291 showing a slight rise in WSE in the hayfield on river left

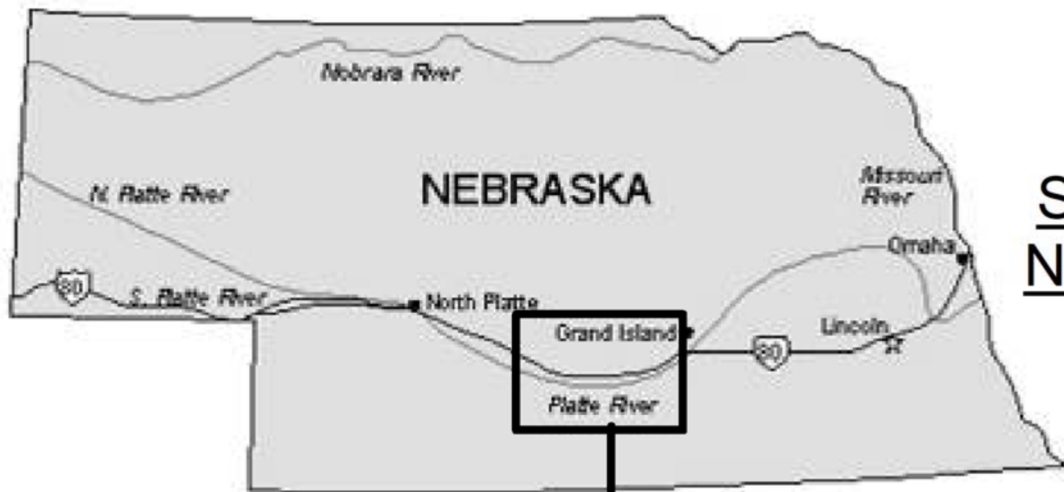
In the upstream most cross section (#106097), the hay field begins at the far left edge of the cross section where the ground surface rises up over 2269 ft; higher than the predicted water surface elevation. In the middle cross section (#104021), the aerial photograph suggests that the field may extend to the water's edge, including the area shown in the cross section to the left of the Main channel. The cross section shows that there is potential for increased flooding in this area immediately adjacent to the stream bank. The aerial photo (Figure 14) near the downstream cross section suggests that there is some buffer between the edge of the field and the main channel stream bank. If this buffer is maintained, flooding may not become any worse in the fields in this area.

SUMMARY

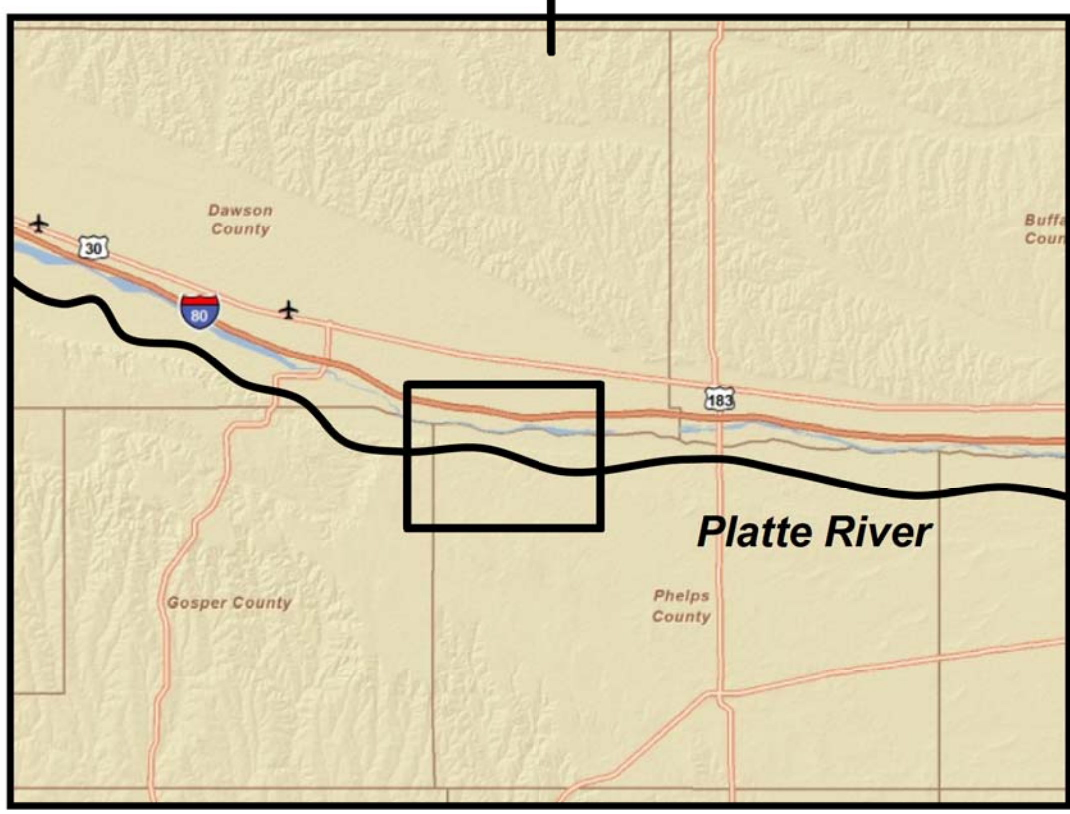
The hydraulic analysis into the feasibility of consolidating flow by routing a portion of the flow in the South channel into the Main channel suggests that the objective of consolidating 85% of the total flow in the Main channel can likely be achieved. The recommended approach to achieving this goal is excavation of a new overflow channel that directs water from the South to the Main channel and creating a partial blockage or blockages in the South channel and in braids around the South channel to create a higher hydraulic head in that channel at the upstream end of the new channel. The investigations described in this report indicate that there may currently be more flow in the Main channel and less in the South channel than the available model suggests. Therefore, the design of the partial blockage system, including determination of the required head increase, will require either updates to the model or an adaptive management approach whereby hydraulic head across the blockage can be modified as necessary to force the desired quantity of water to the Main channel.

REFERENCES

Barnes, H.H., Jr., 1967, *Roughness characteristics of natural channels:* U.S. Geological Survey Water-Supply Paper 1849, 213 p.



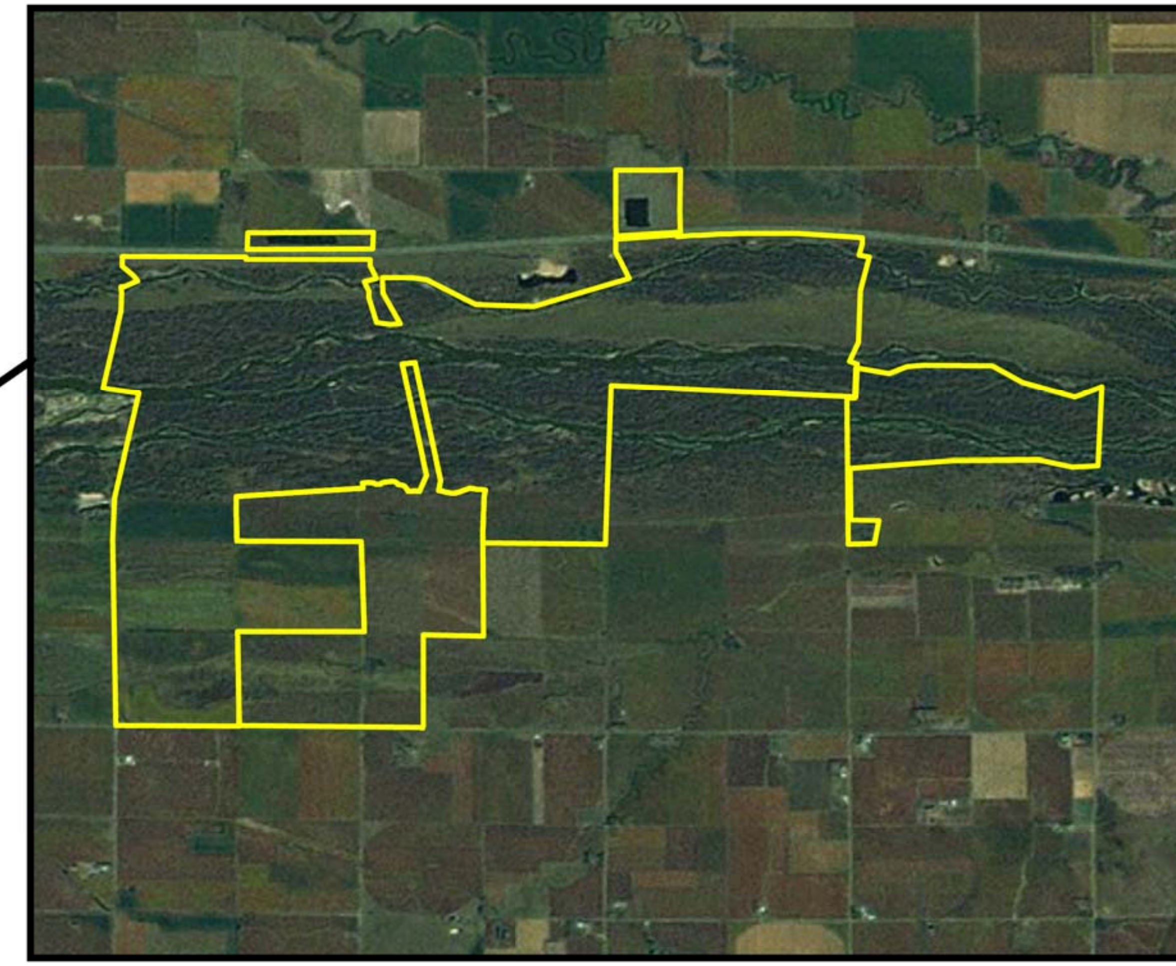
STATE OF NEBRASKA



Vicinity Map

Site Location

Latitude 40.681382°
Longitude -99.461741°



Site Map

Cottonwood Ranch Property

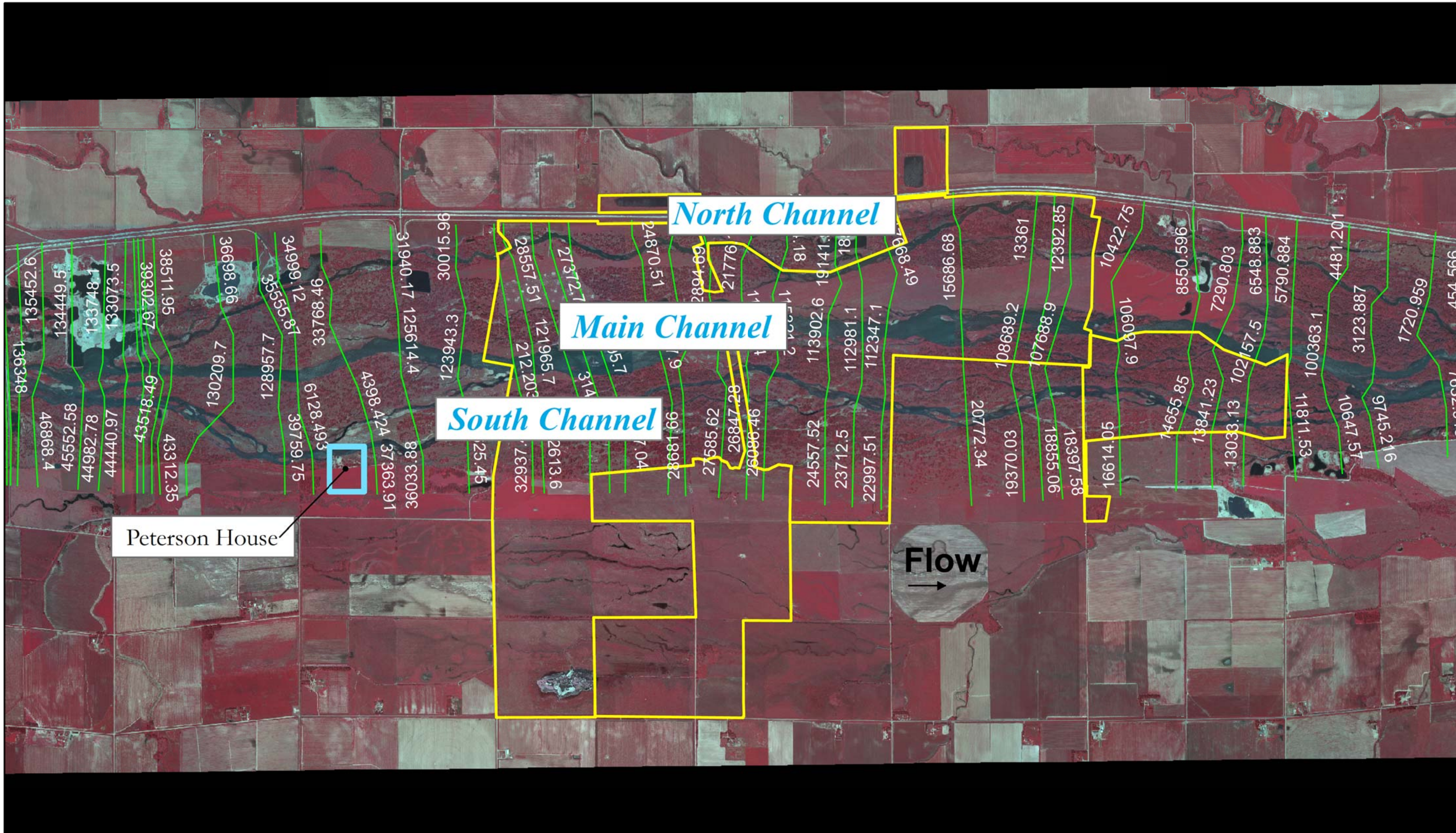
HEC Geo-RAS Output Maps for Cottonwood Ranch Flow Consolidation Investigation

inter·fluve inc.
3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property

SHEET
1 of 21

0 2,650 5,300 10,600 Feet



inter·fluve

3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Site Overview

SHEET

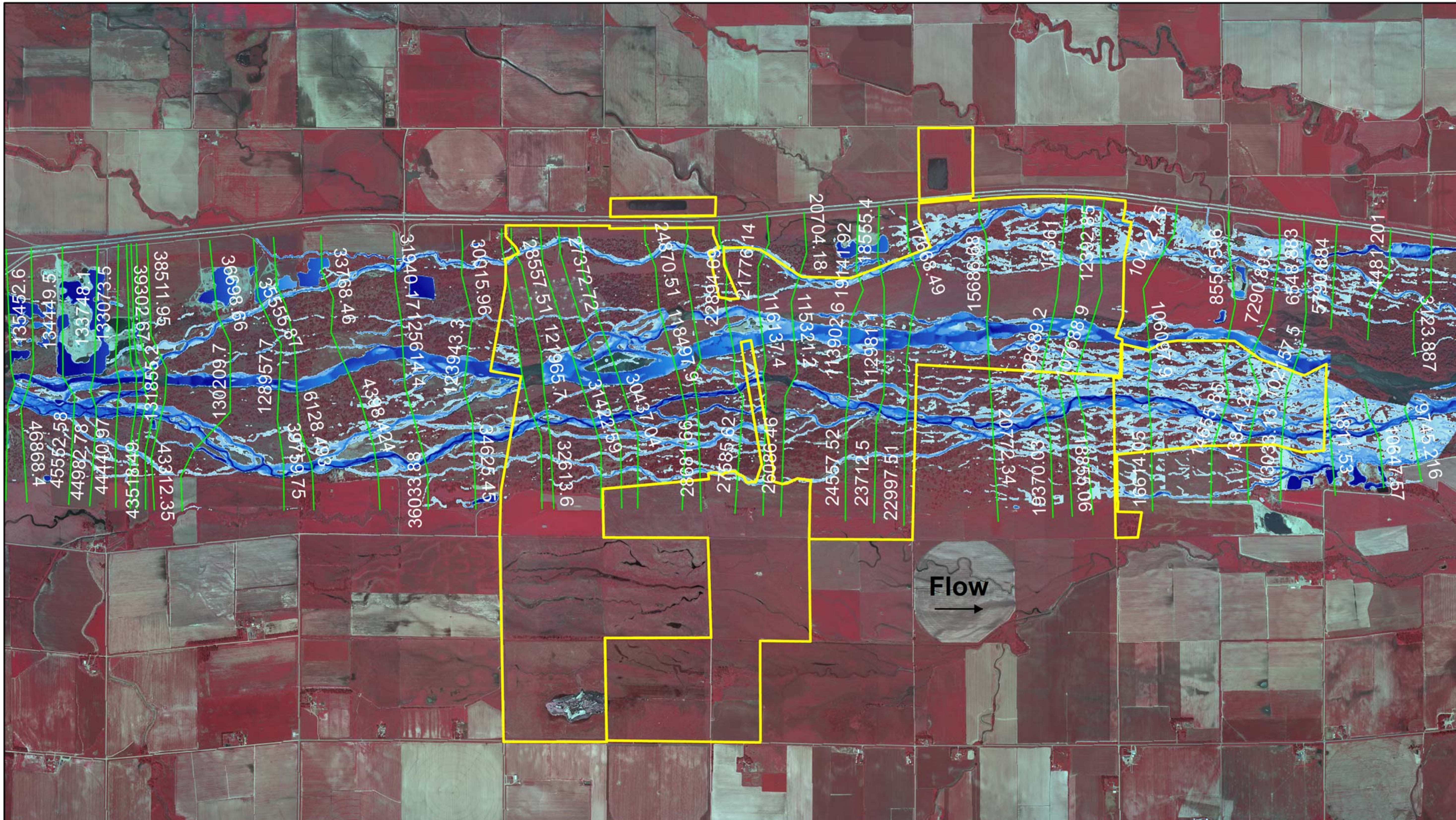
2 of 21

0 1,200 2,400 4,800 Feet

Property Boundary

River Cross-Sections

N



inter·fluve
3602 Atwood Ave
Suite 3
Madison, WI 53714

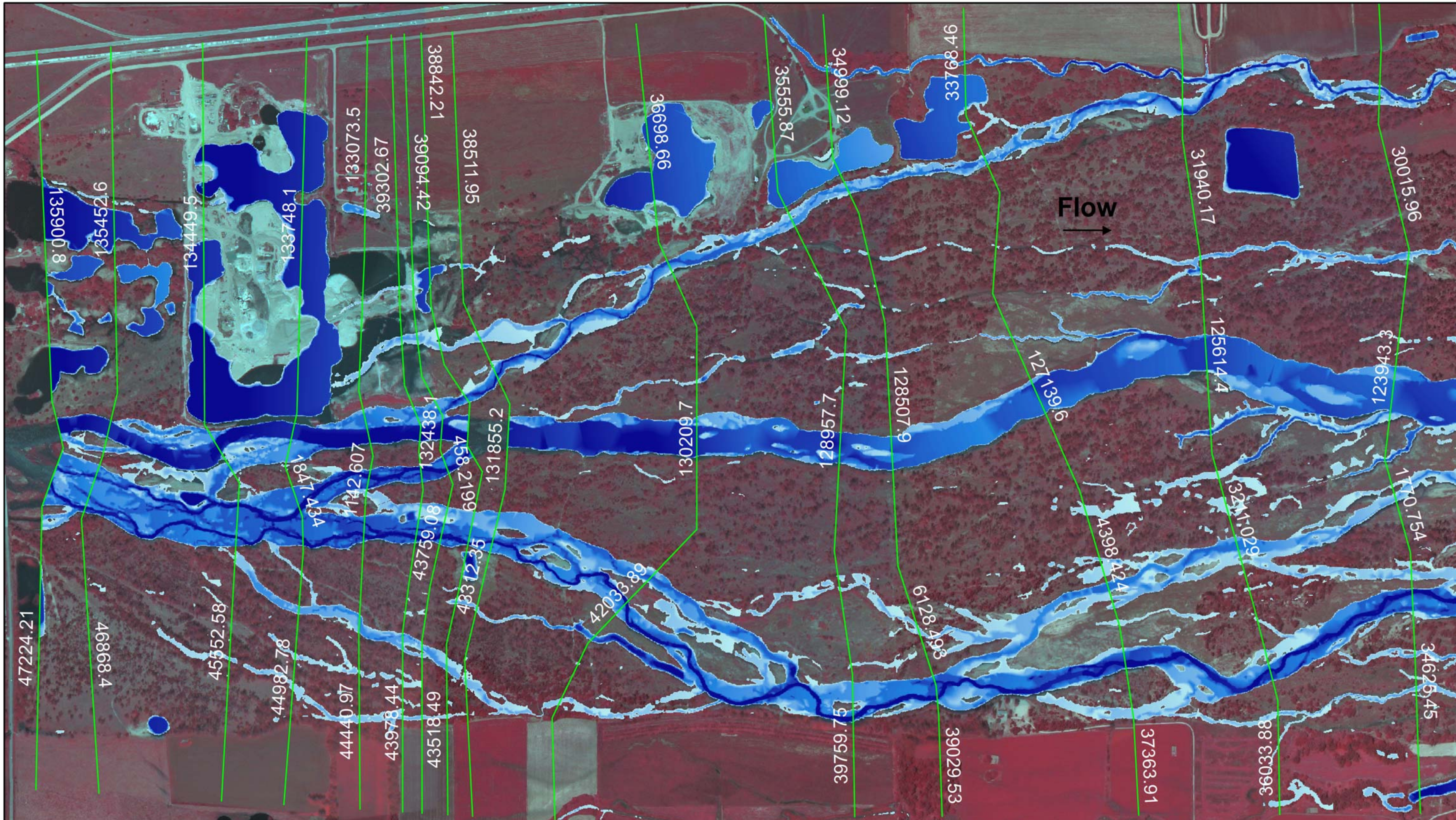
Platte River
Cottonwood Ranch Property
Existing Conditions at 7000cfs
Site Overview

SHEET
3 of 21

Depth (ft) at 7000 cfs
High : 6.32
Low : 0.1

0 1,050 2,100 4,200 Feet

N



inter·fluve

3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Existing Conditions at 7000cfs
Zoom 1
Just Upstream of Cottonwood Ranch Property

SHEET

4 of 21

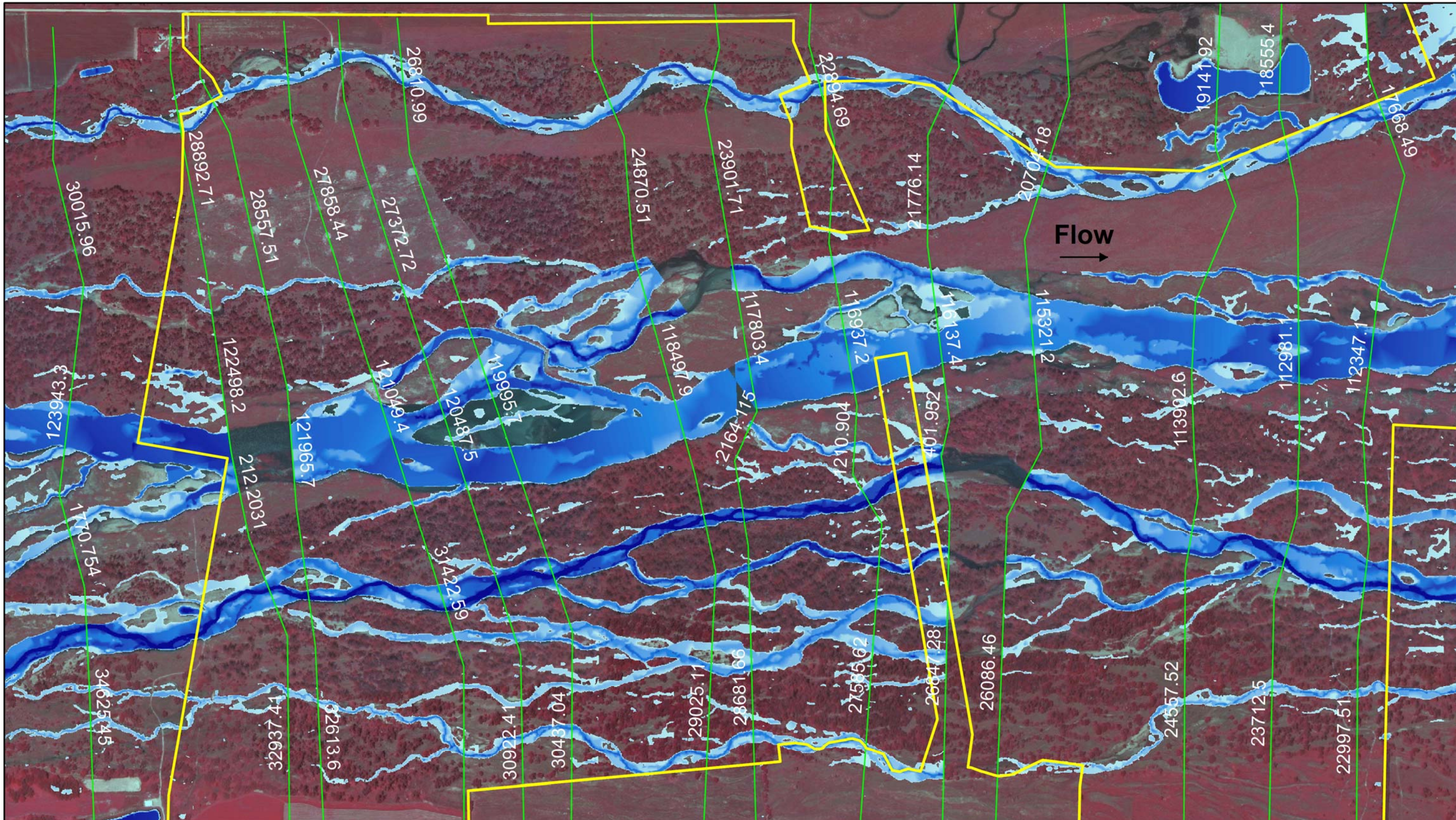
Depth (ft) at 7000 cfs

High : 6.32

Low : 0.1

0 355 710 1,420 Feet

N



inter·fluve

3602 Atwood Ave
Suite 3
Madison, WI 53714

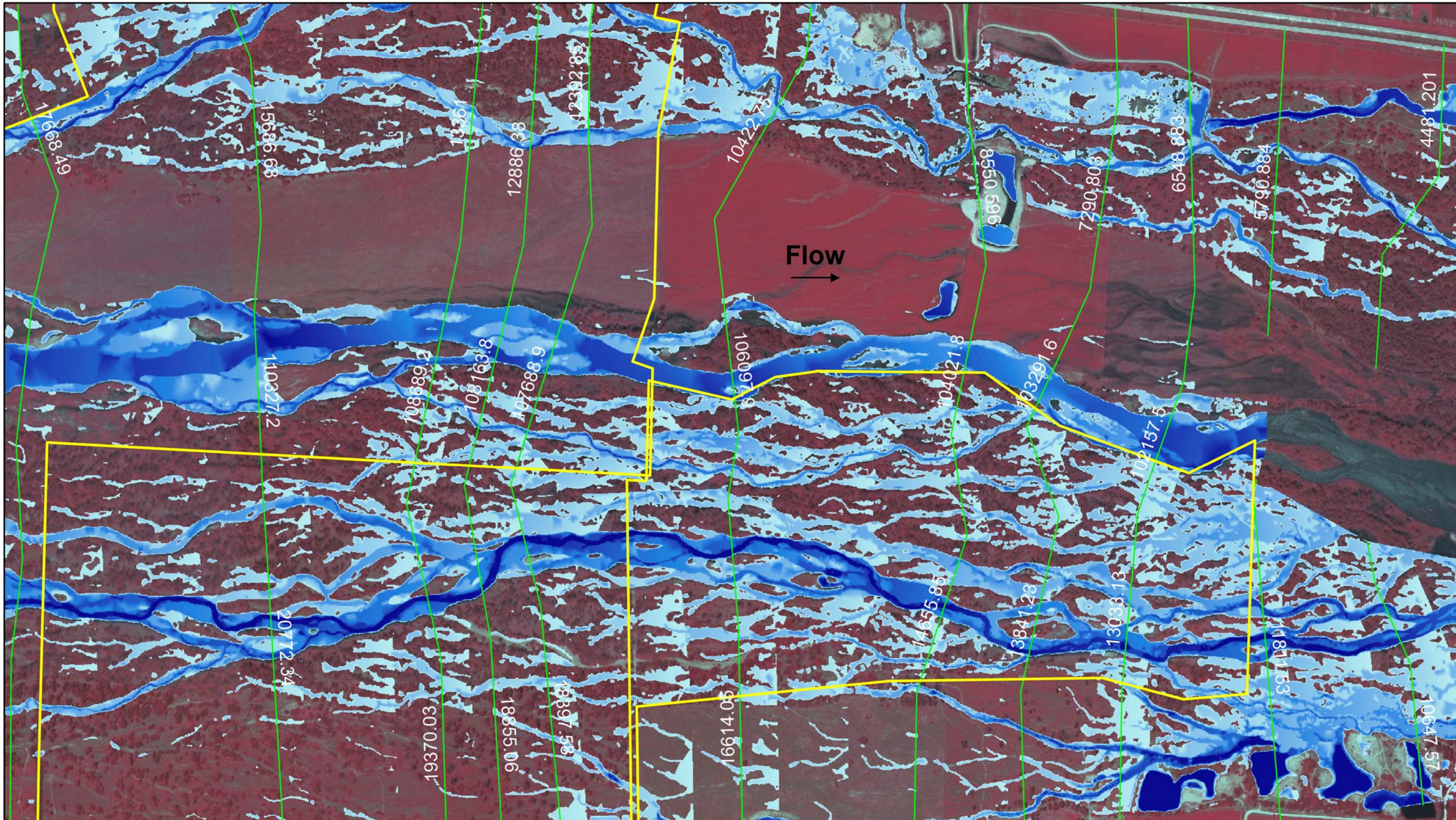
Platte River
Cottonwood Ranch Property
Existing Conditions at 7000cfs
Zoom 2

SHEET
5 of 21

Depth (ft) at 7000 cfs
High : 6.32
Low : 0.1

0 355 710 1,420 Feet

N



3602 Atwood Ave
Suite 3
Madison, WI 53714

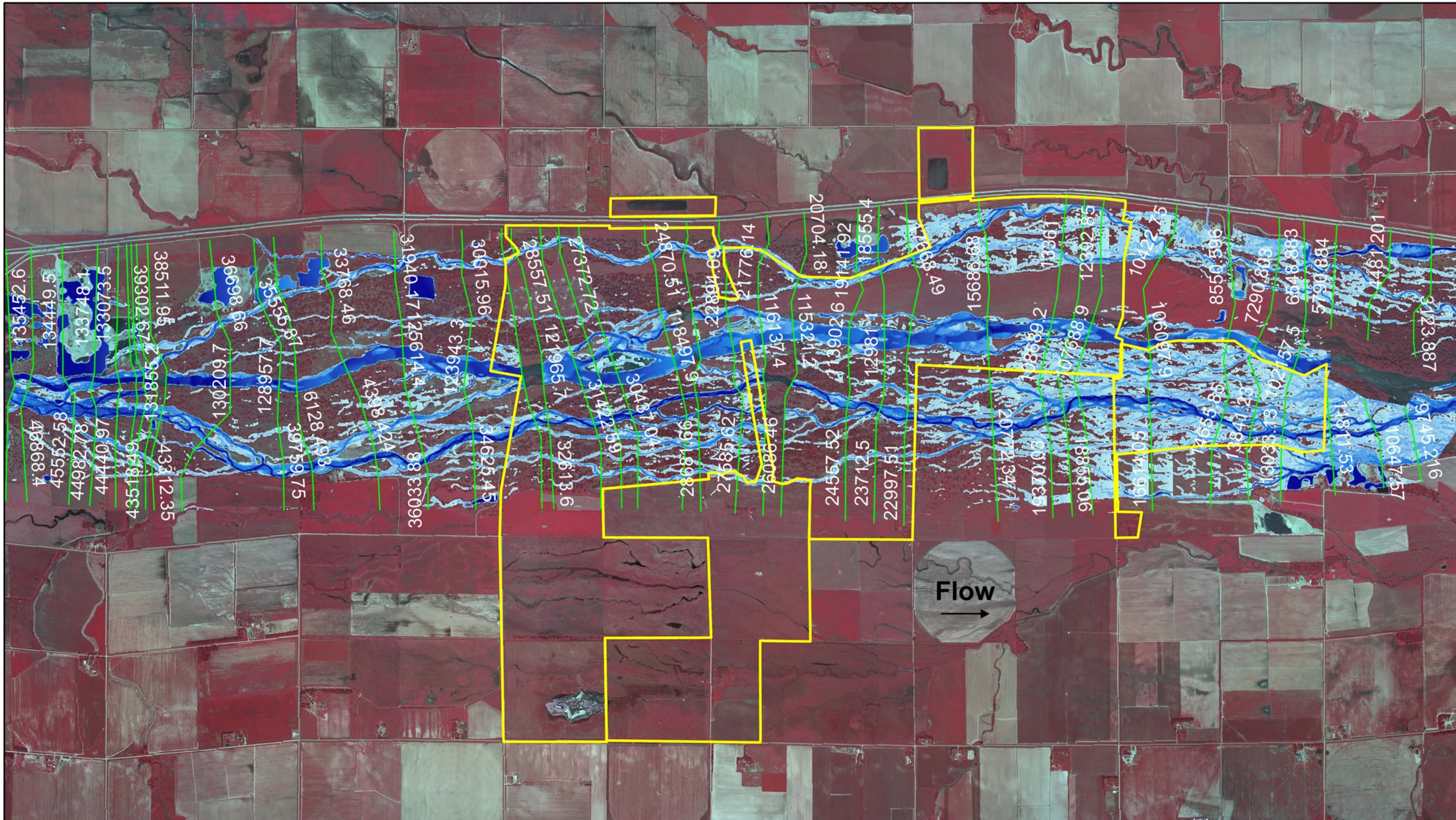
Platte River
Cottonwood Ranch Property
Existing Conditions at 7000cfs
Zoom 3

SHEET
6 of 21

Depth (ft) at 7000 cfs
High : 6.32
Low : 0.1

0 355 710 1,420 Feet

N



inter·fluve

3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Existing Conditions at 8000cfs
Site Overview

SHEET

7 of 21

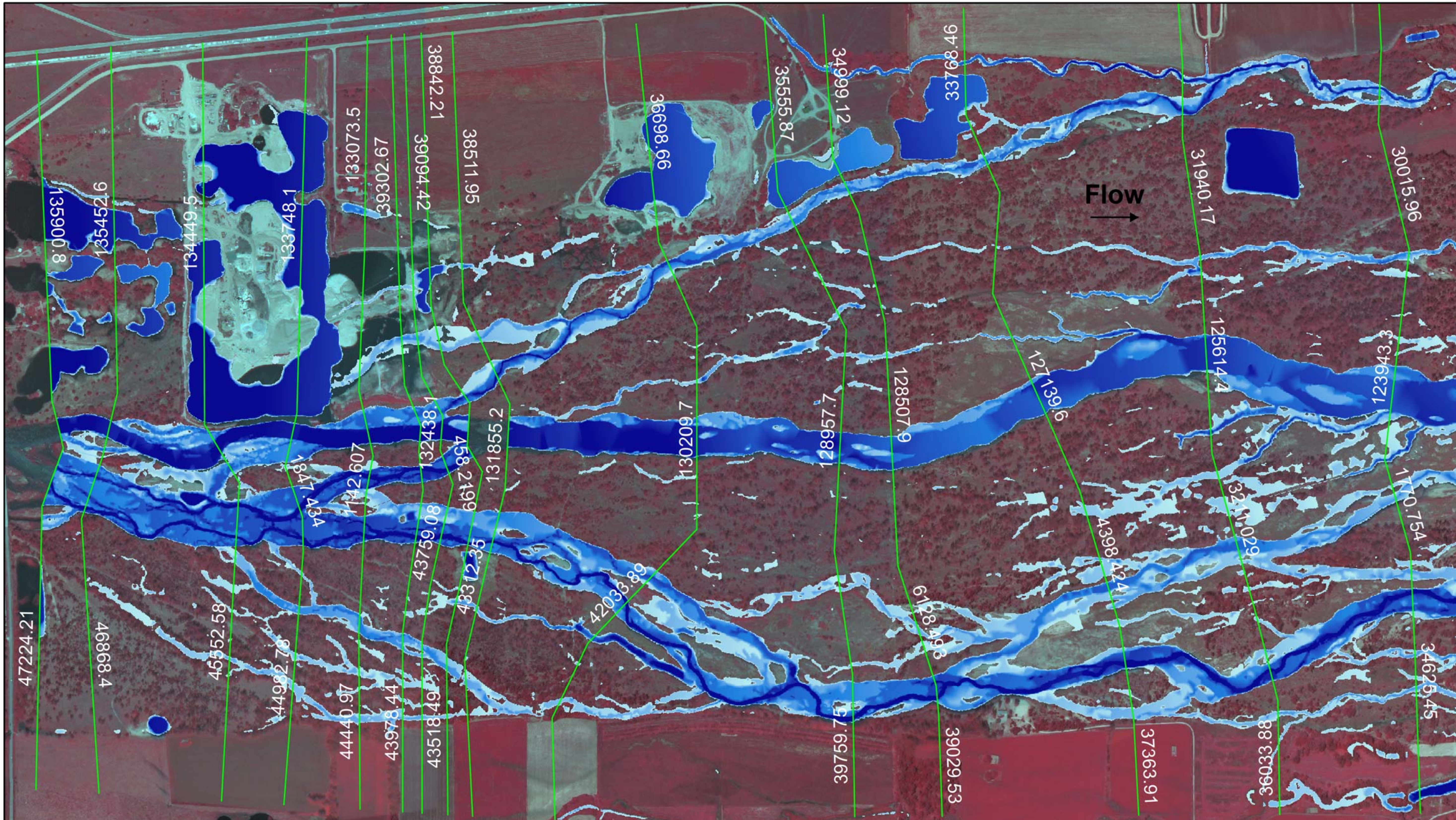
Depth (ft) at 8000 cfs

High : 6.69

Low : 0.1

0 1,050 2,100 4,200 Feet

N



3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Existing Conditions at 8000cfs
Zoom 1
Just Upstream of Cottonwood Ranch Property

SHEET

8 of 21

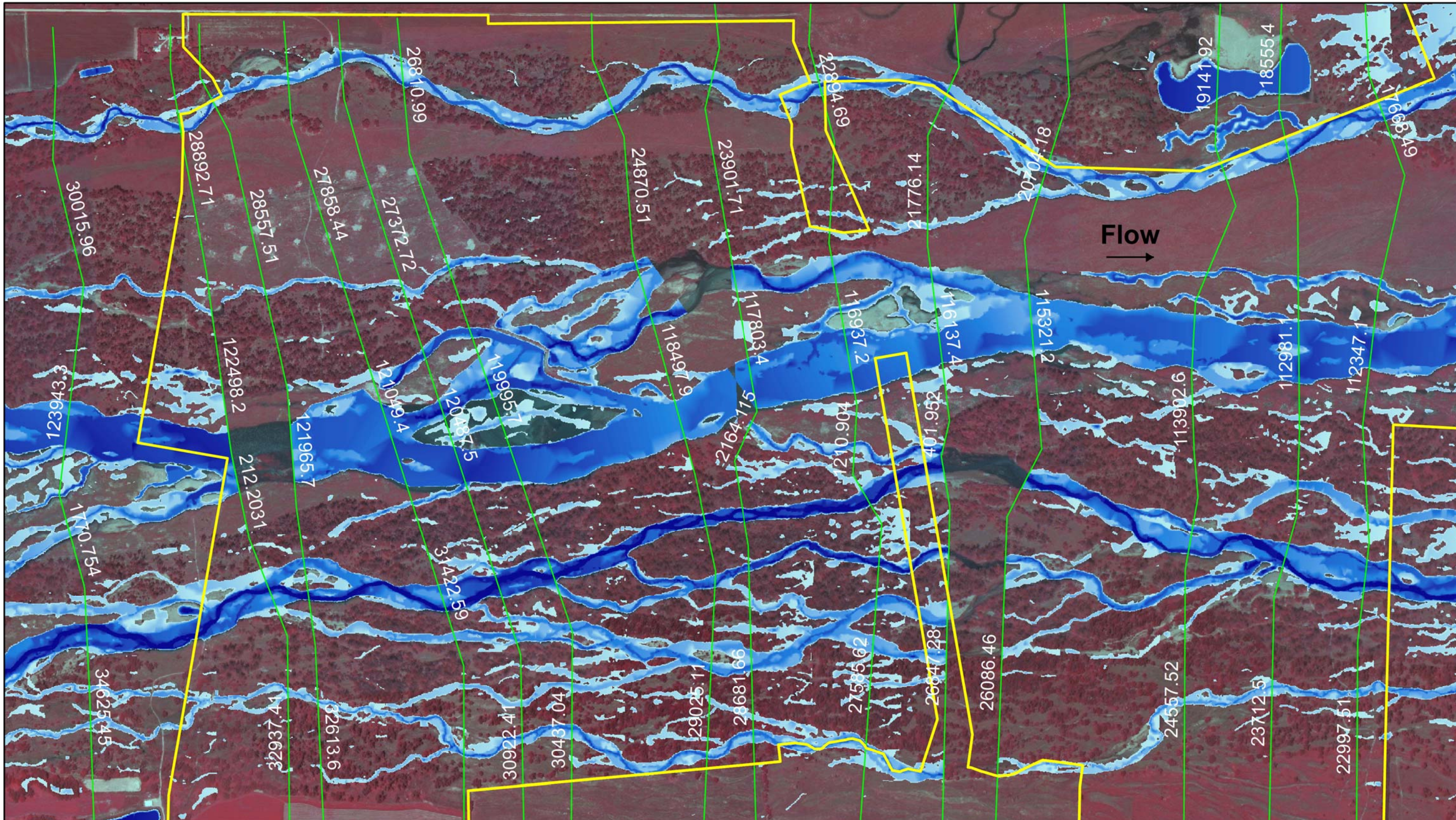
Depth (ft) at 8000 cfs

High : 6.69

Low : 0.1

0 355 710 1,420 Feet

N



3602 Atwood Ave
Suite 3
Madison, WI 53714

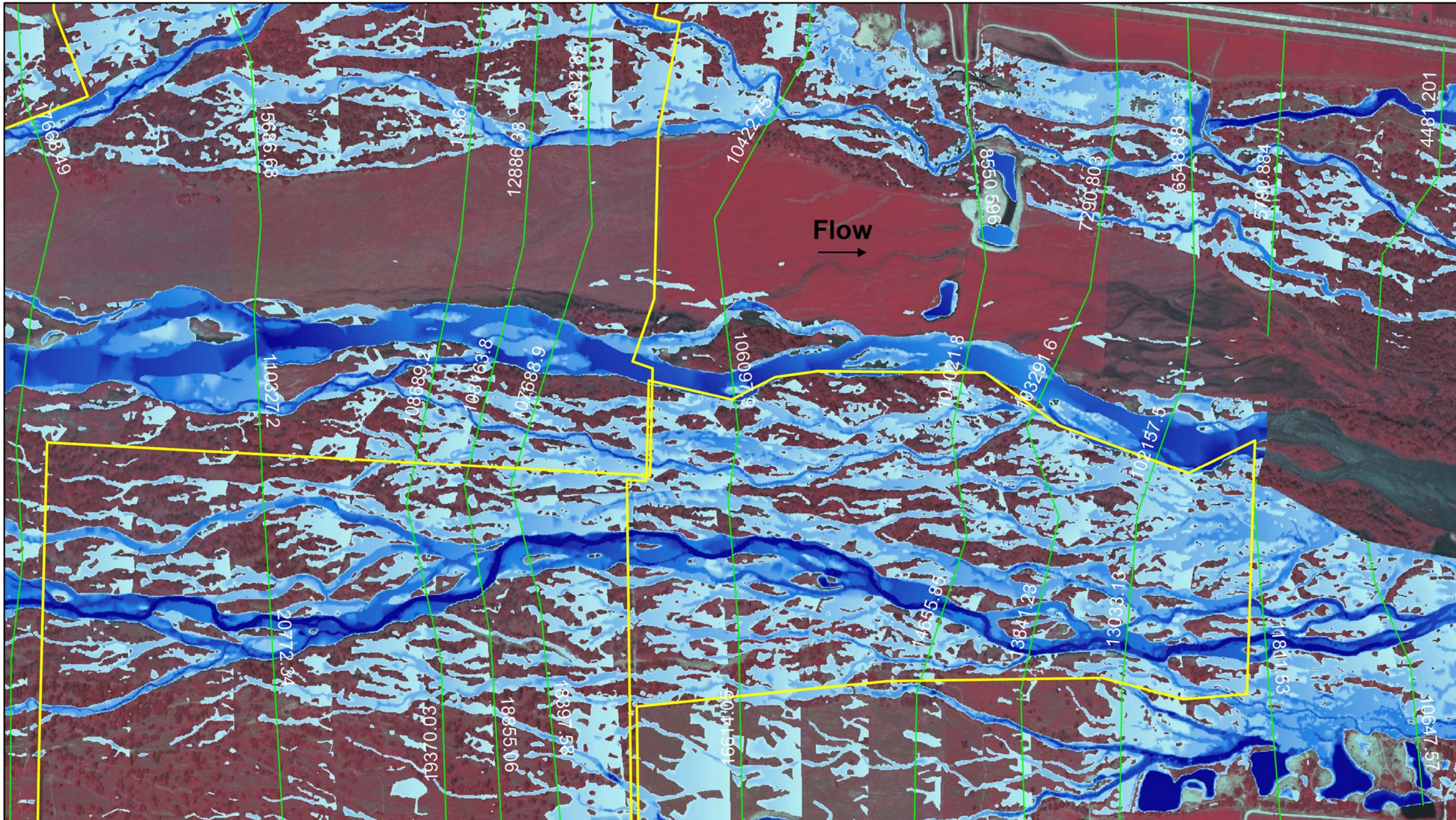
Platte River
Cottonwood Ranch Property
Existing Conditions at 8000cfs
Zoom 2

SHEET
9 of 21

Depth (ft) at 8000 cfs
High : 6.69
Low : 0.1

0 355 710 1,420 Feet

N



3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Existing Conditions at 8000cfs
Zoom 3

SHEET

10 of 21

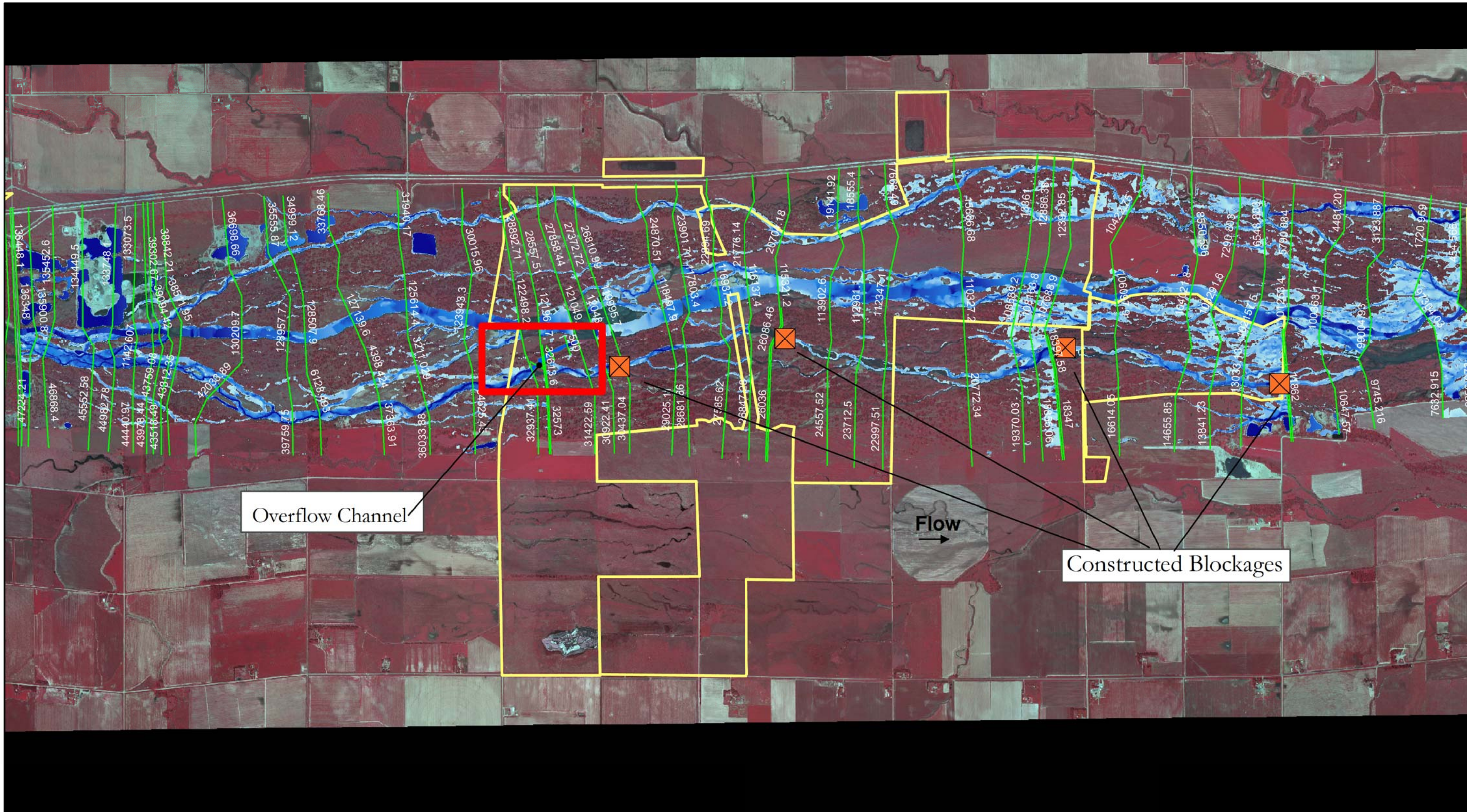
Depth (ft) at 8000 cfs

High : 6.69

Low : 0.1

0 355 710 1,420 Feet

N



inter·fluve

3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Scenario 4
Overflow Channel Upstream &
Multiple Hydraulic Controls

SHEET

11 of 21

Depth (ft) at 8000 cfs

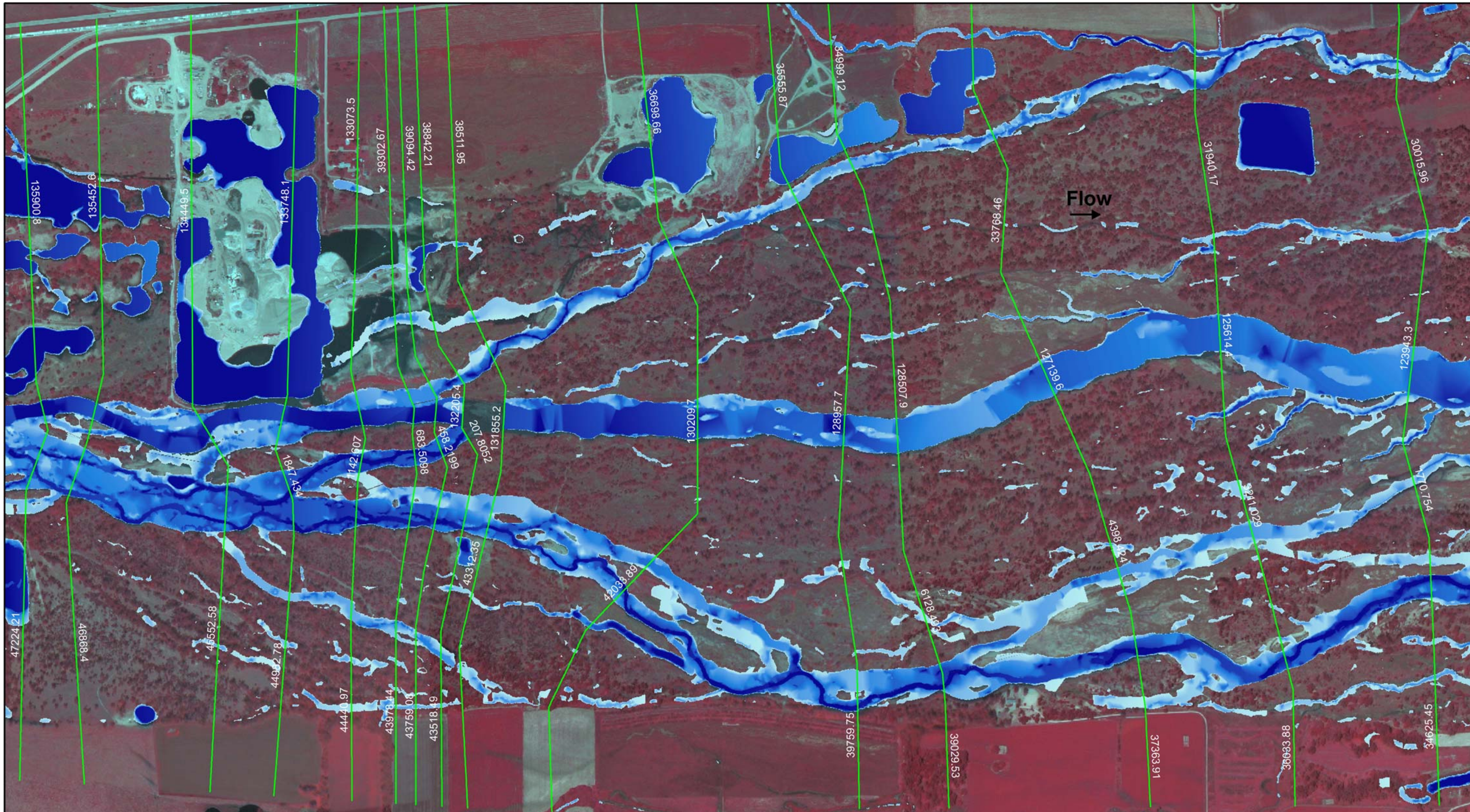
High : 6.7

Low : 0.1

0 1,100 2,200 4,400 Feet

Blockages

N



inter·fluve

3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Scenario 4 Zoom1:
Overflow Channel Upstream &
Multiple Hydraulic Controls

SHEET

12 of 21

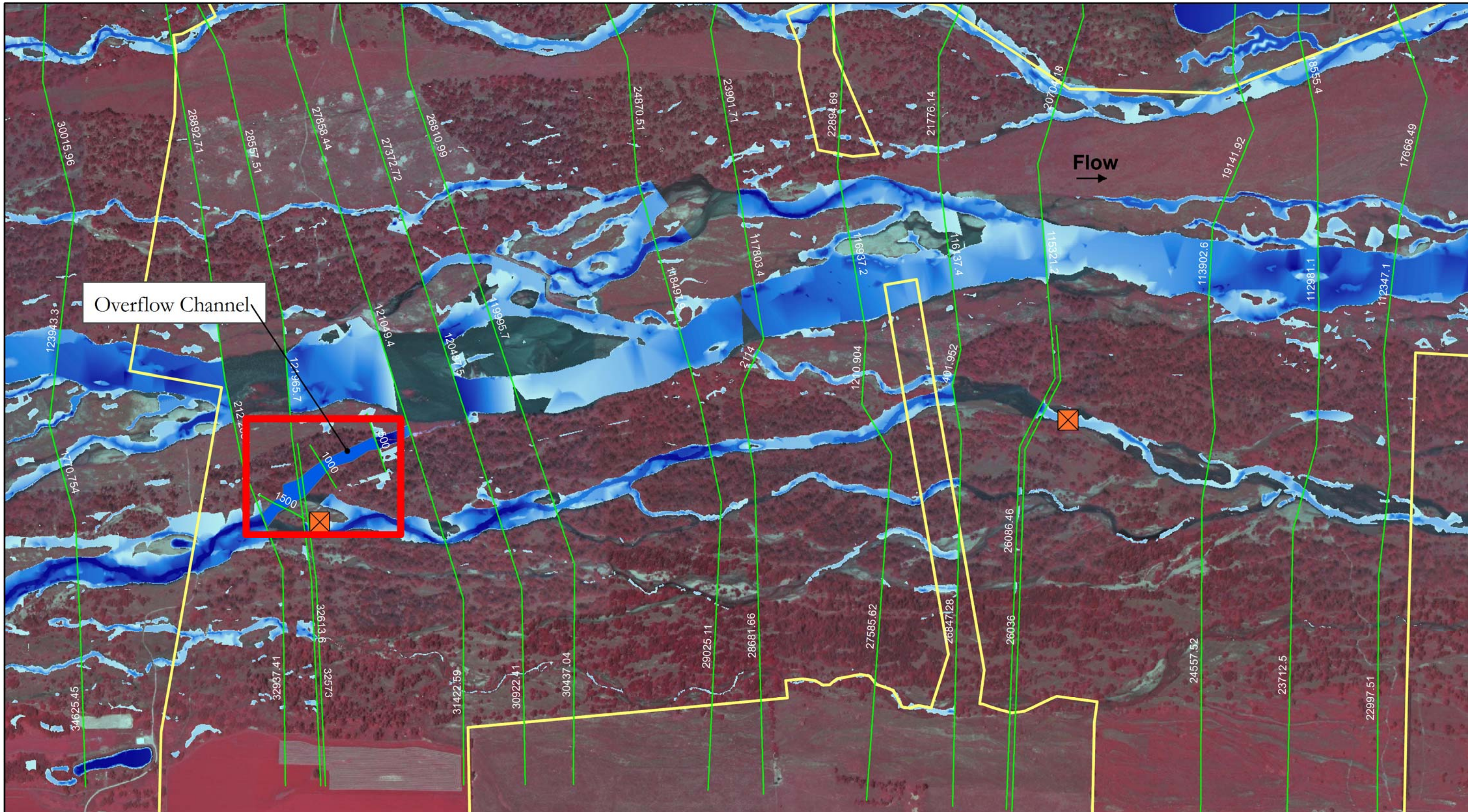
Depth (ft) at 8000 cfs

High : 6.7

Low : 0.1

0 345 690 1,380 Feet

N



inter·fluve

3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Scenario 4 Zoom 2:
Overflow Channel Upstream &
Multiple Hydraulic Controls

SHEET

13 of 21

Depth (ft) at 8000 cfs

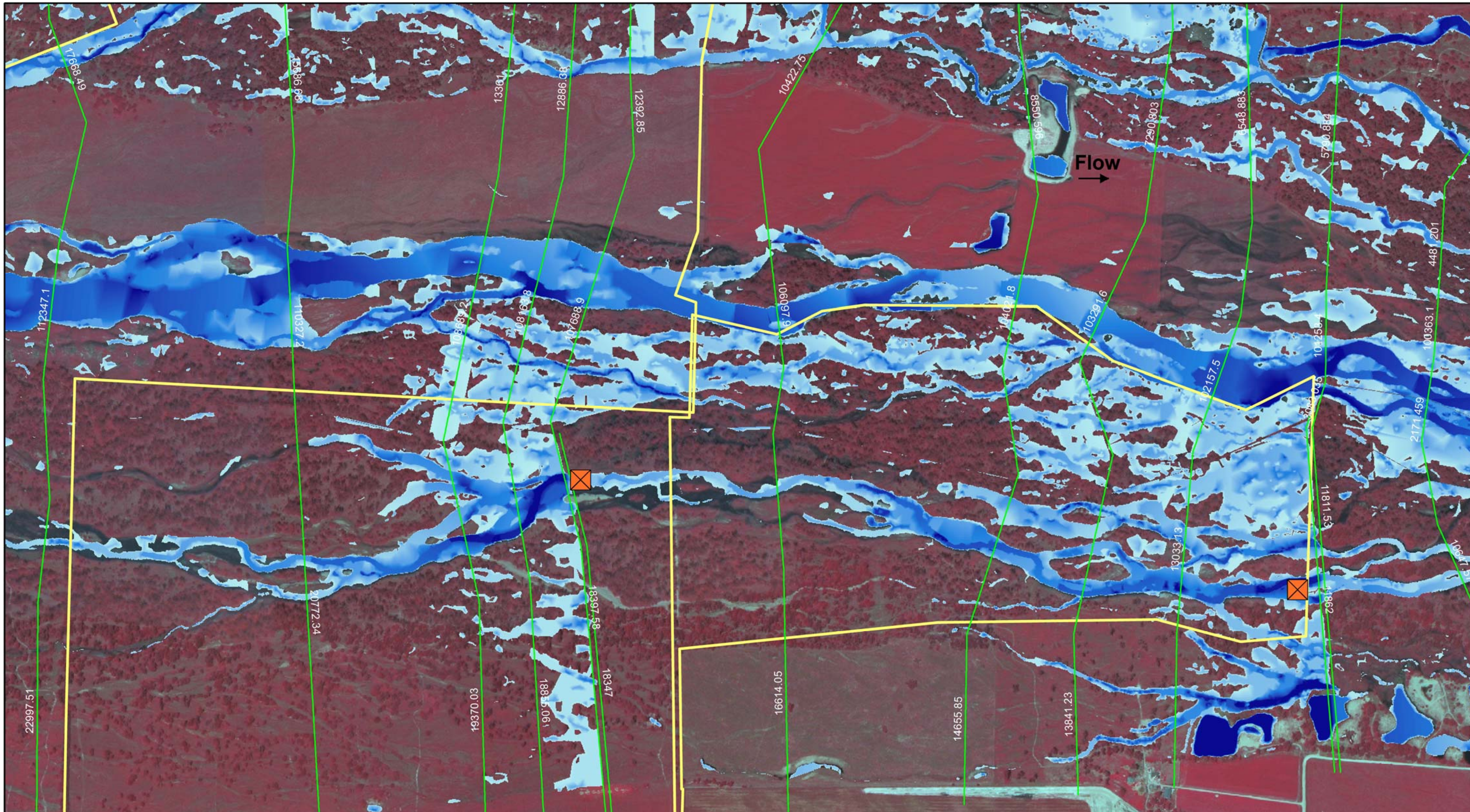
High : 6.7

Low : 0.1

0 350 700 1,400 Feet

Blockages

N



3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Scenario 4 Zoom 3:
Overflow Channel Upstream &
Multiple Hydraulic Controls

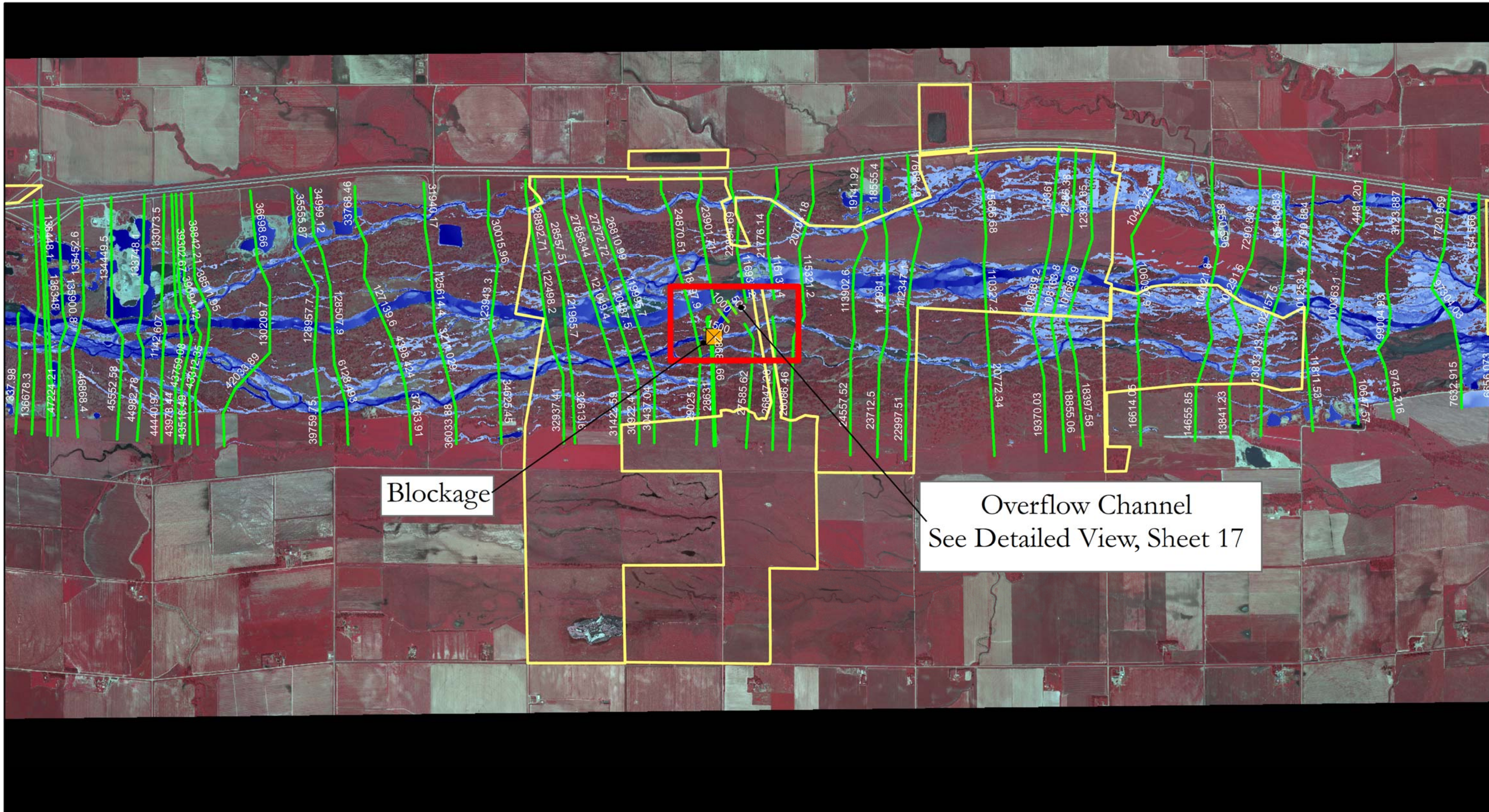
SHEET
14 of 21

Depth (ft) at 8000 cfs
High : 6.7
Low : 0.1

0 350 700 1,400 Feet

Blockages

N



3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Scenario 5:
Overflow Channel Downstream &
Single Hydraulic Controls

Depth (ft) at 8000 cfs

Value

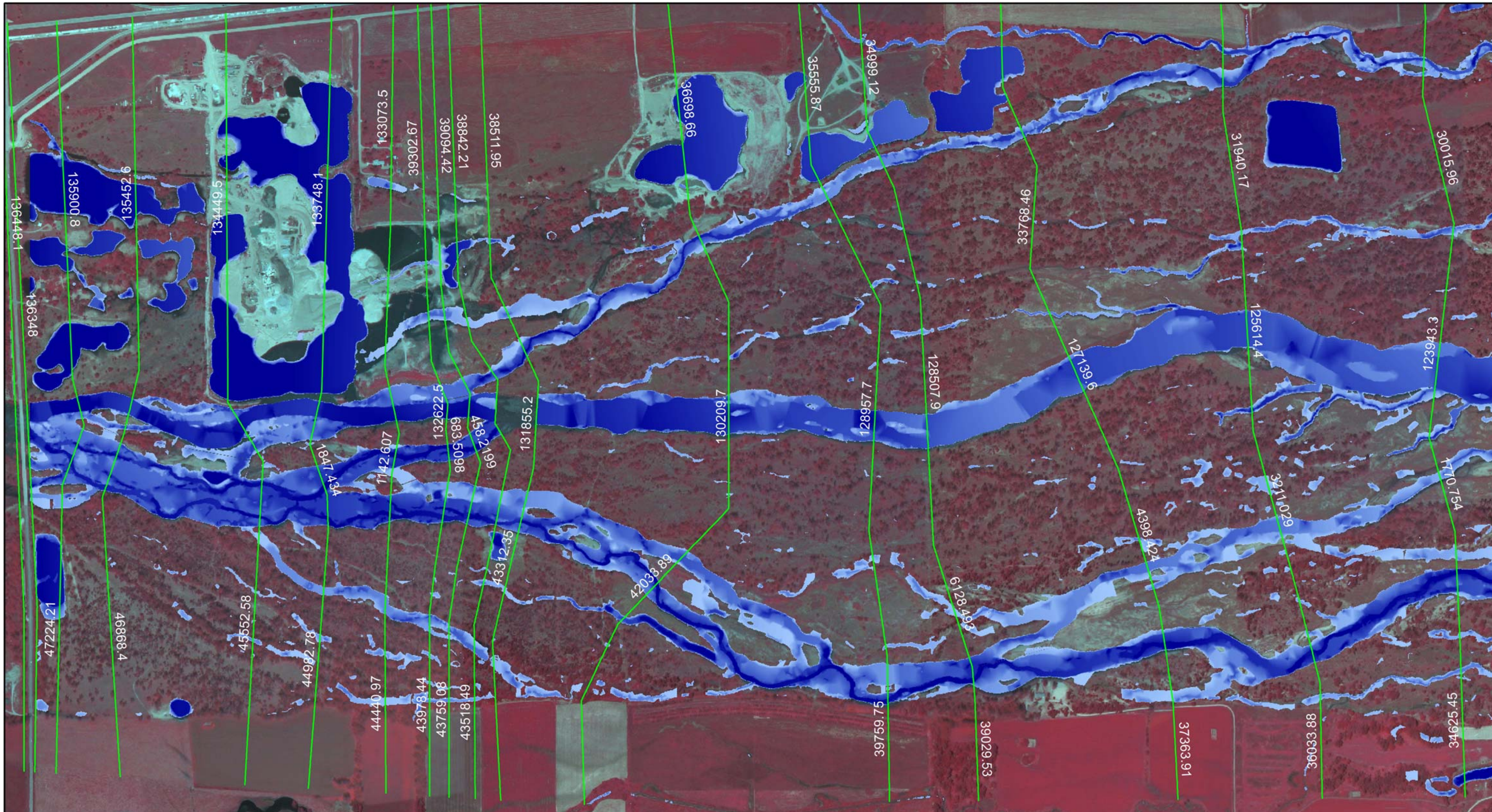
High : 6.69

Low : 0.02

0 1,150 2,300 4,600 Feet

Blockage

N



3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Scenario 5 Zoom 1:
Overflow Channel Downstream &
Single Hydraulic Controls

SHEET

16 of 21

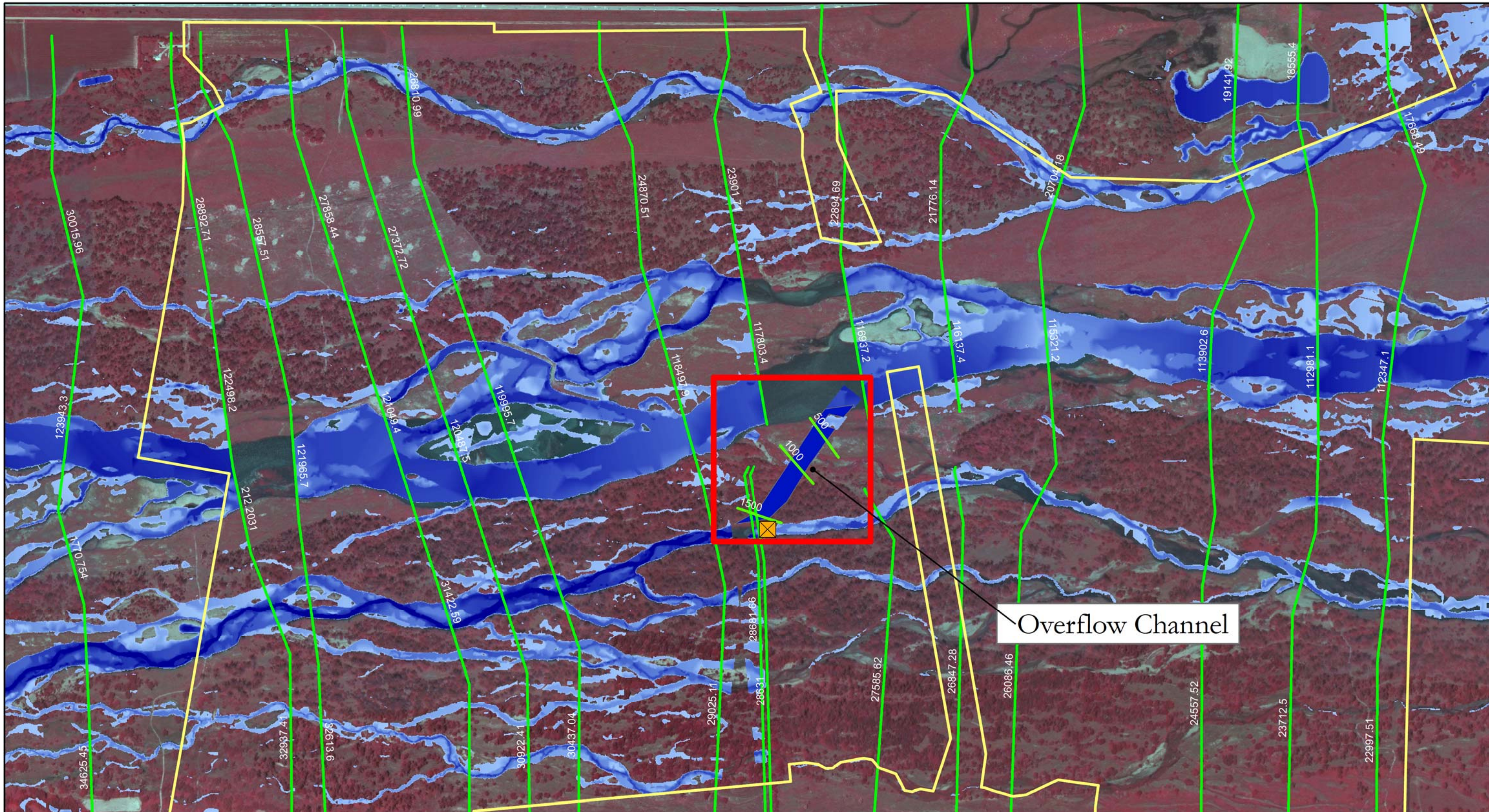
Depth (ft) at 8000cfs

High : 6.7

Low : 0.1

0 430 860 1,720 Feet

N



3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Scenario 5 Zoom 2:
Overflow Channel Downstream &
Single Hydraulic Controls

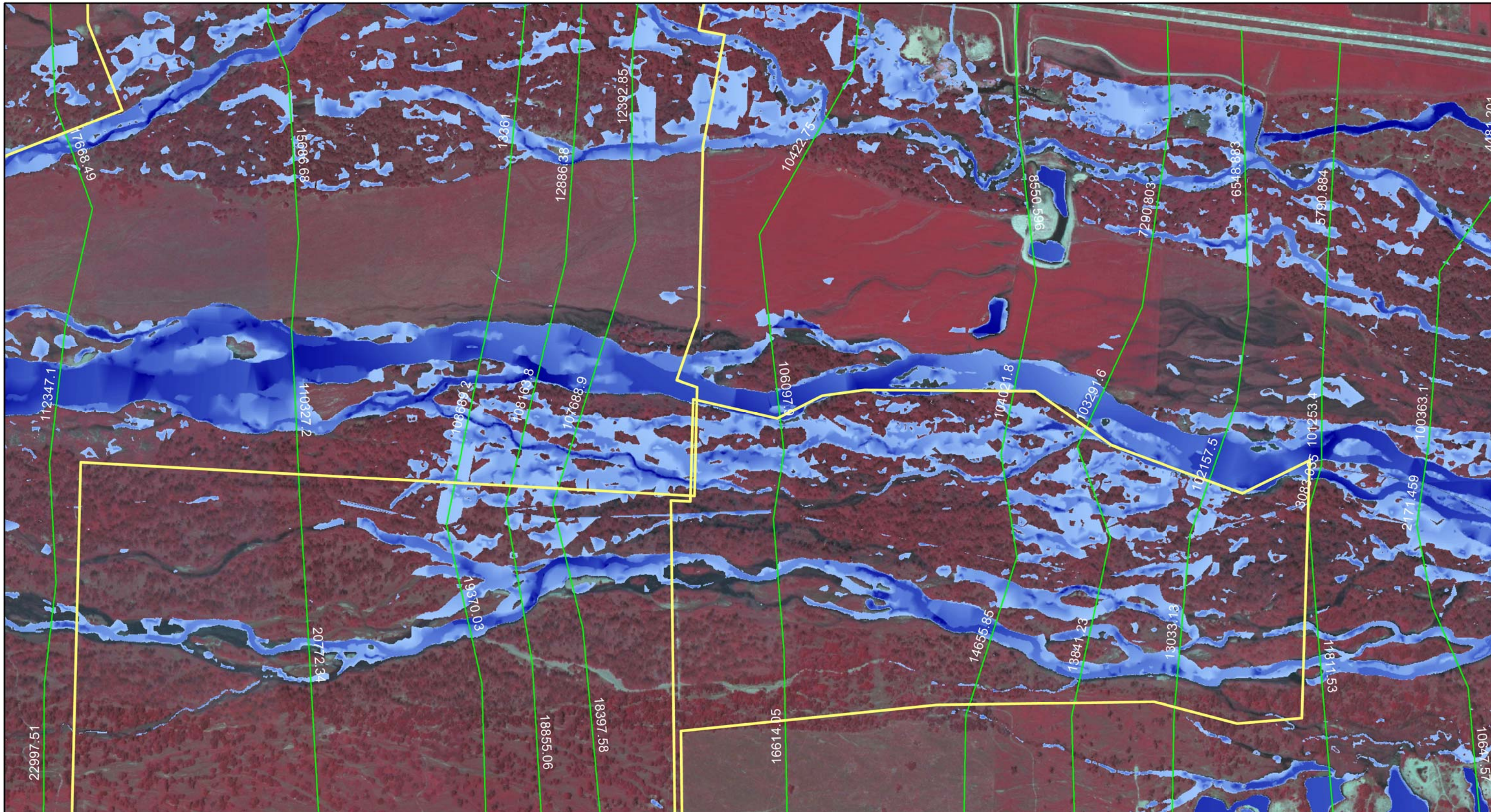
SHEET
17 of 21

Depth (ft) at 8000 cfs
Value
High : 6.69
Low : 0.02

Blockage

0 365 730 1,460 Feet

N



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Platte River
Cottonwood Ranch Property
Scenario 5 Zoom 3:
Overflow Channel Downstream &
Single Hydraulic Controls

SHEET

18 of 21

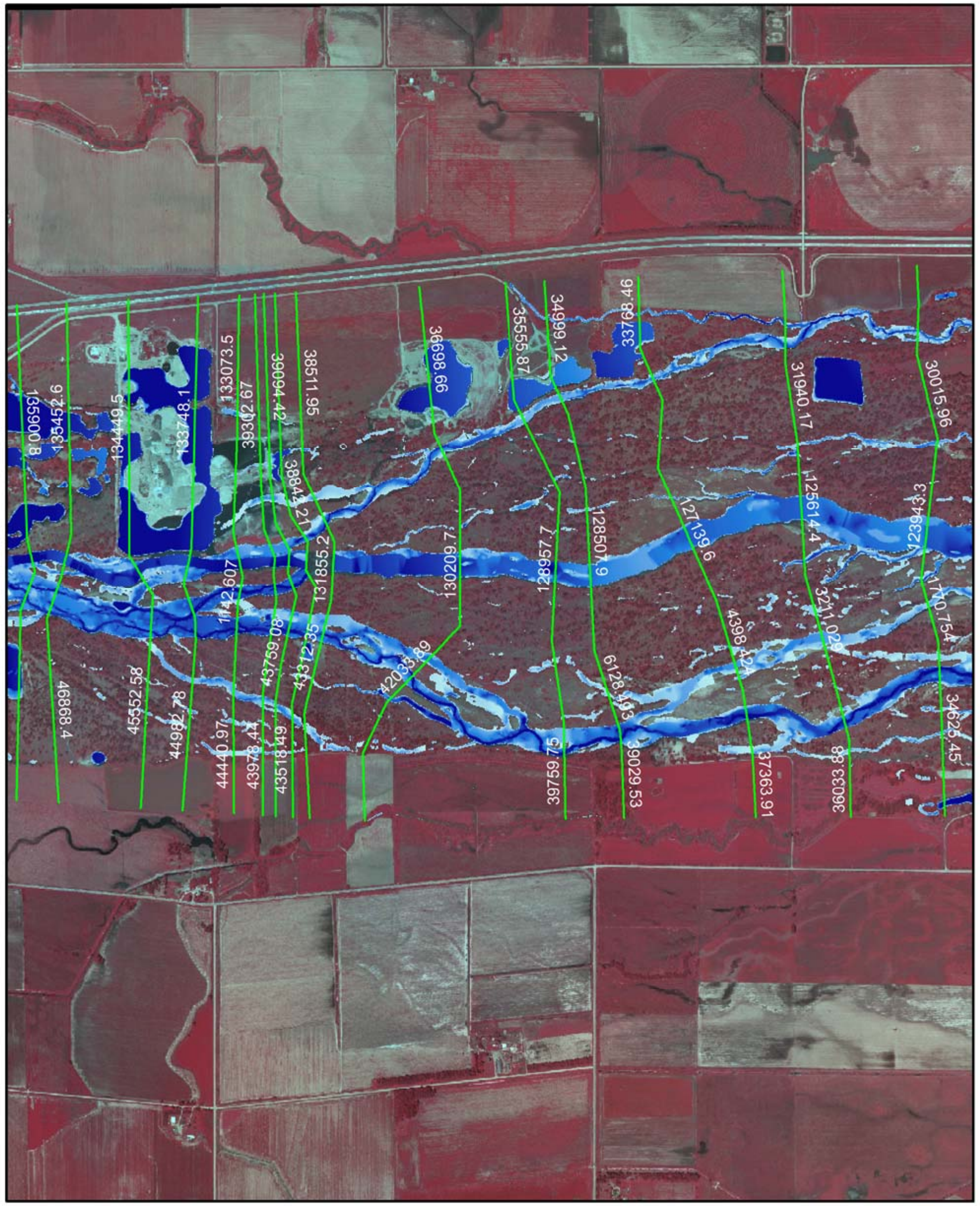
Depth (ft) at 8000cfs

High : 6.7

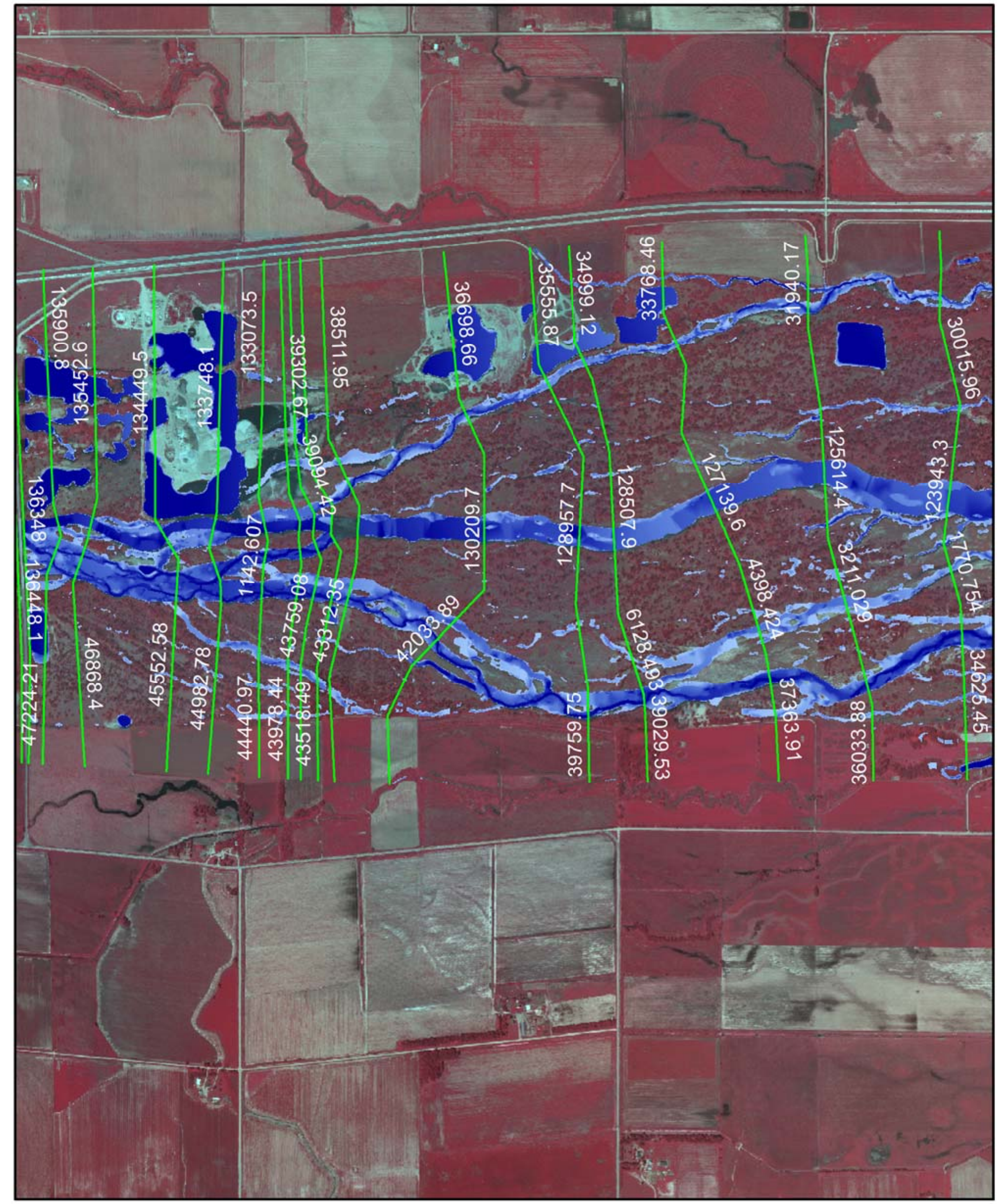
Low : 0.1

0 345 690 1,380 Feet

N



Scenario 4



Scenario 5

inter·fluve inc.

3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Scenario 4 and 5 Comparison
Zoom 1
Just upstream of Cottonwood Ranch Property

SHEET
19 of 21

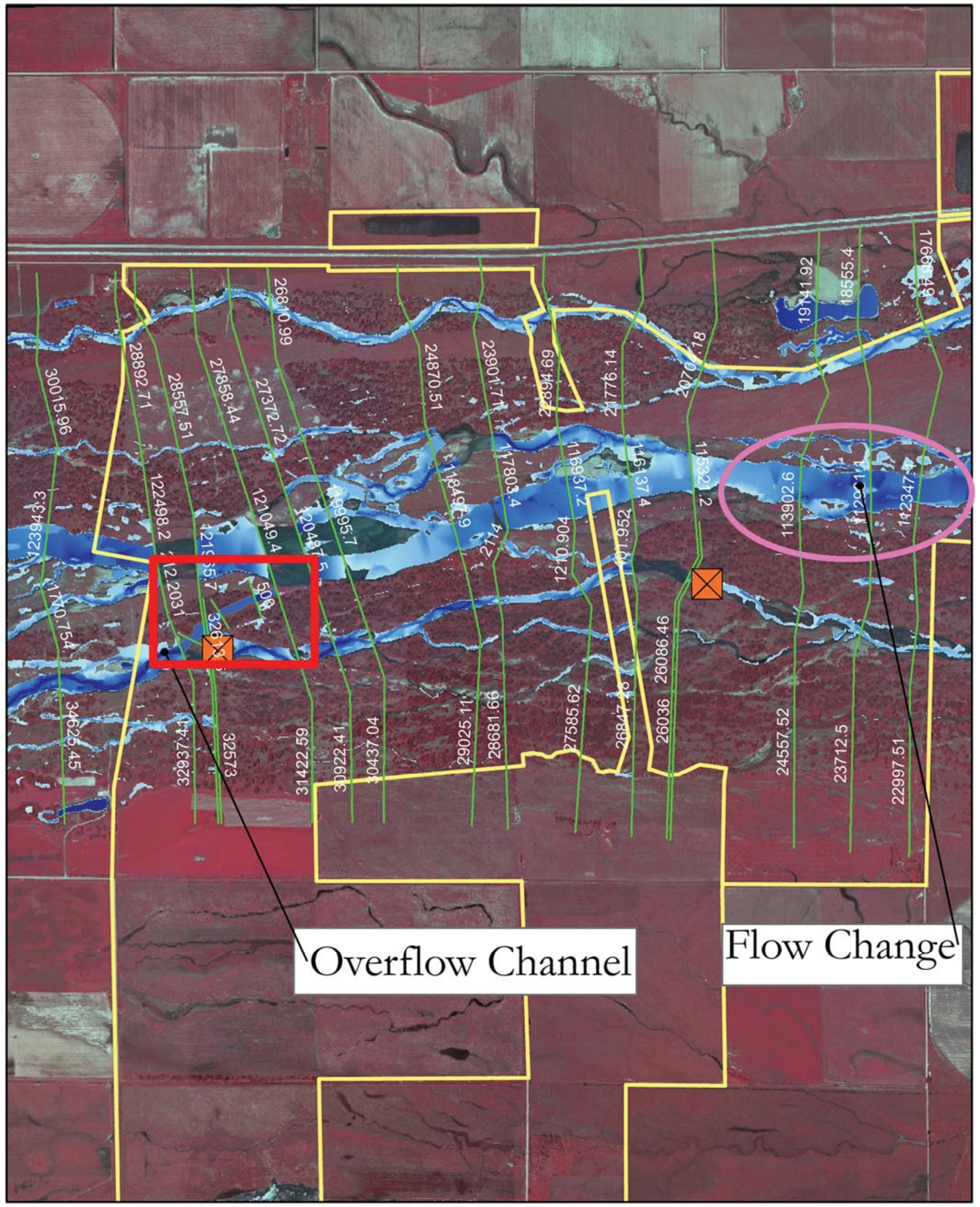
Depth (ft) at 8000cfs

High : 6.7

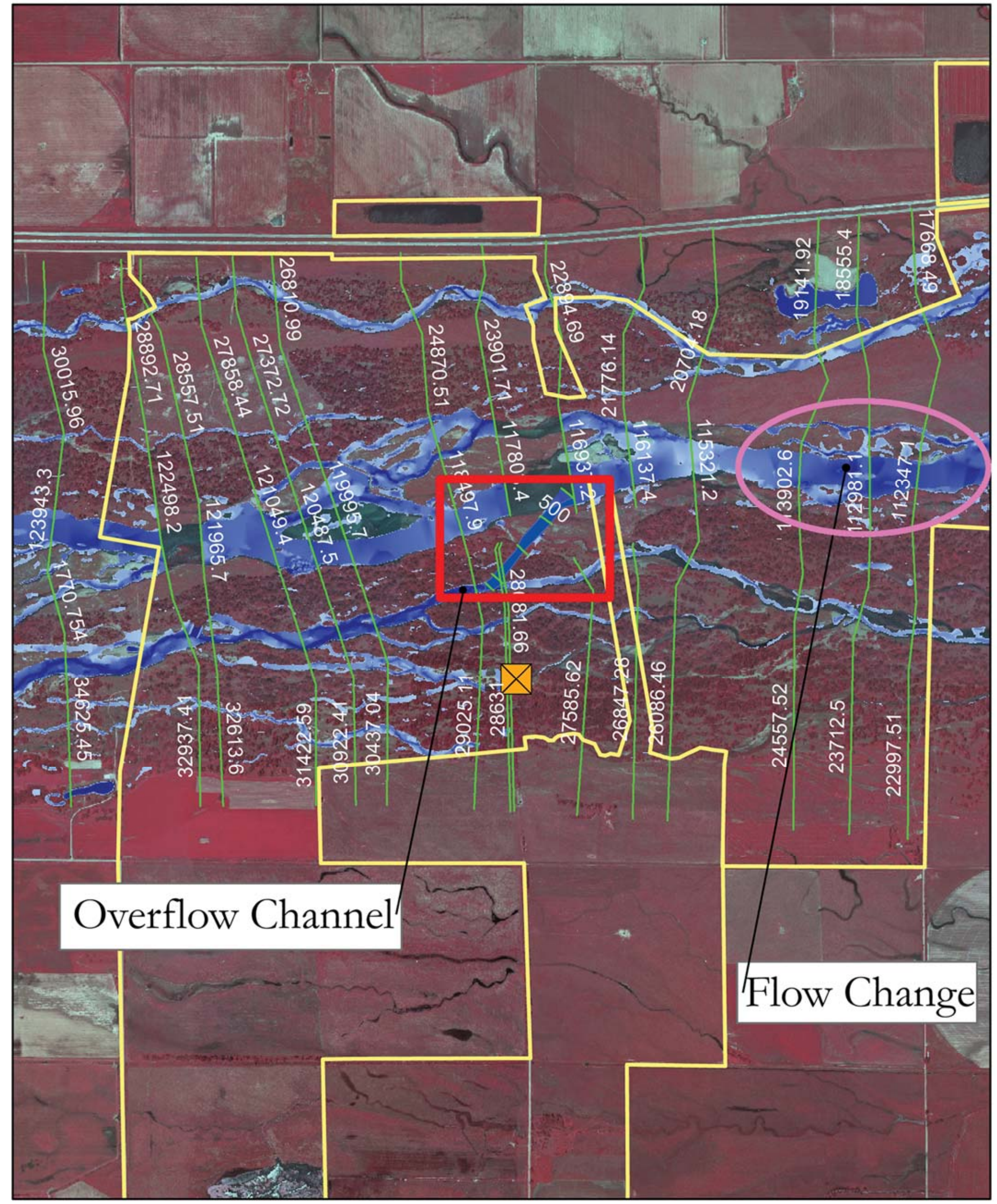
Low : 0.1

0 800 1,600 3,200 Feet

N



Scenario 4



Scenario 5

inter·fluve inc.

3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Scenario 4 and 5 Comparison
Zoom 2

SHEET
20 of 21

Depth (ft) at 8000cfs

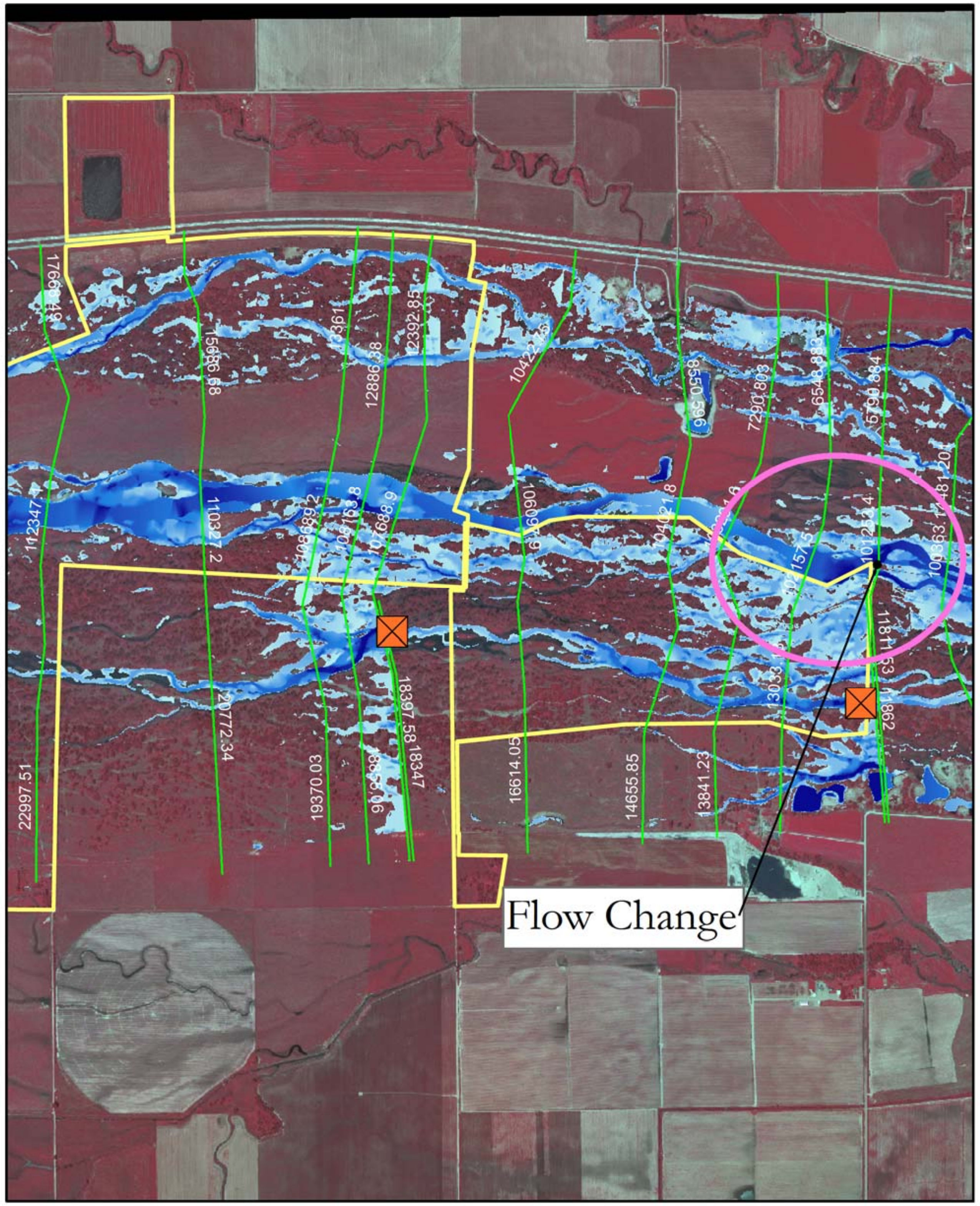
High : 6.7

Low : 0.1

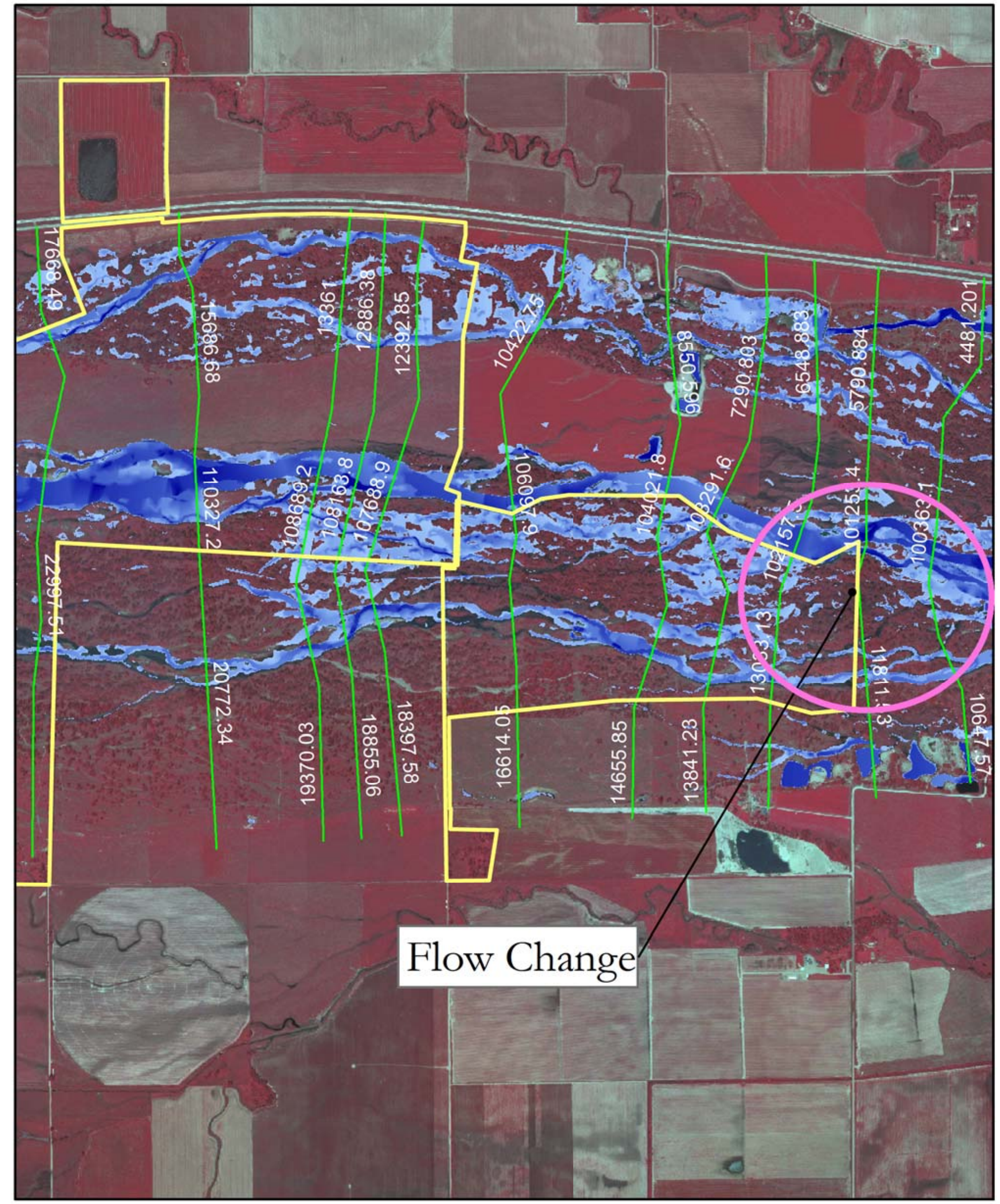
0 800 1,600 3,200 Feet

Blockage

N



Scenario 4



Scenario 5

inter·fluve inc.

3602 Atwood Ave
Suite 3
Madison, WI 53714

Platte River
Cottonwood Ranch Property
Scenario 4 and 5 Comparison
Zoom 3

SHEET

21 of 21

Depth (ft) at 8000cfs

High : 6.7

Low : 0.1

0 800 1,600 3,200 Feet

Blockage

N

APPENDIX C – Unsteady Flow Investigation Report

INTRODUCTION

The steady-state flow model was used in HEC-RAS for evaluation of flow consolidation at Cottonwood Ranch. However, the use of the steady-state model had to be justified considering the design flows from the upstream reservoir will arrive in unsteady pulses. To demonstrate the applicability of the steady-state model, constant flow rates were routed through the Platte River using UNET, the unsteady flow model within HEC-RAS. The output from this model was compared to the results from the steady-state flow simulation in HEC-RAS to verify the use of the steady-state model for design.

The unsteady flow model solves the momentum and continuity equation simultaneously at each cross section at every time step. As a result of this solution scheme, the momentum method can more accurately describe the distribution of the flows in a network with one or more parallel channels or loops, consistent with the multiple channels and junction at Cottonwood Ranch. The following paragraphs summarize the results of the existing unsteady flow model, and compares them with the steady flow model.

EXISTING CONDITIONS MODEL

The existing conditions model was used to determine the attenuation of peak flows. Hydrographs at the upstream station in the hydraulic model (at North Platte, NE; river mile 310) could not be compared directly with the hydrograph at the downstream end of the model (at Chapman, NE; river mile 156) because numerous canals and tributaries enter the river.

For instance, the Johnson 2 Power Return provides a significant amount of flow immediately upstream of Cottonwood Ranch This inflow is upstream of the proposed overflow channel location and the discharge pattern is highly irregular due to flow regulation (Figure 1). Discharges vary in a step-wise fashion at this inflow and drastically alter the upstream hydrograph (Figure 2). The resultant hydrograph at Overton, NE (river mile 240) is significantly different from the initial hydrograph at the upstream end of the modeled reach (Figure 3).

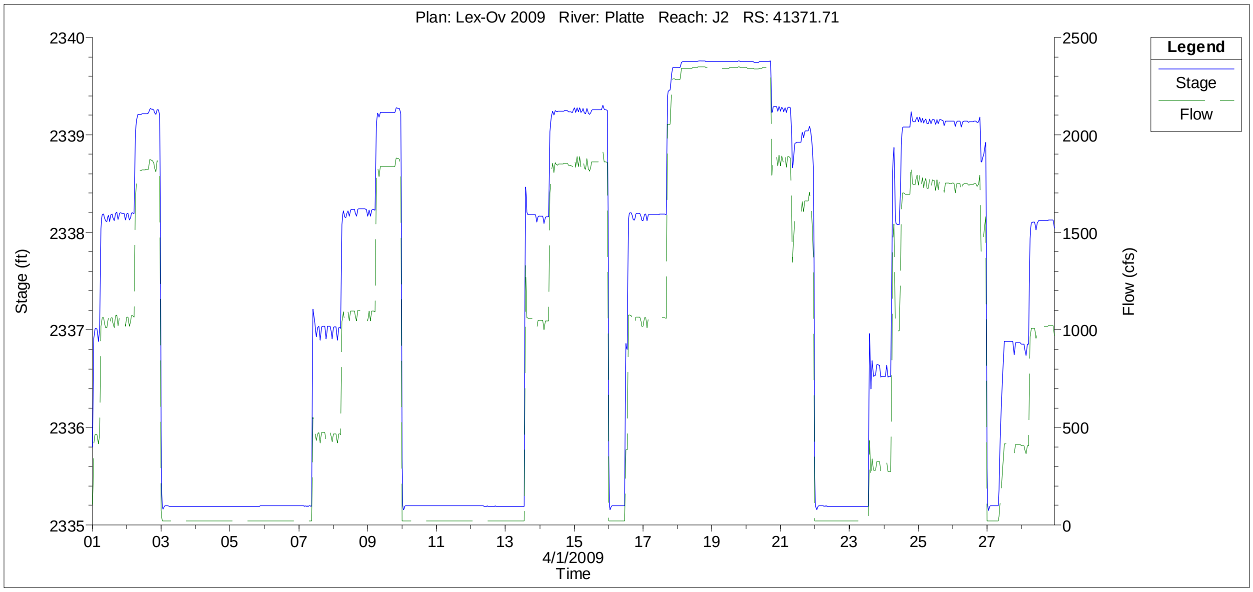


Figure 1. Hydrograph at the outflow of reach J2, a regulated flow return.

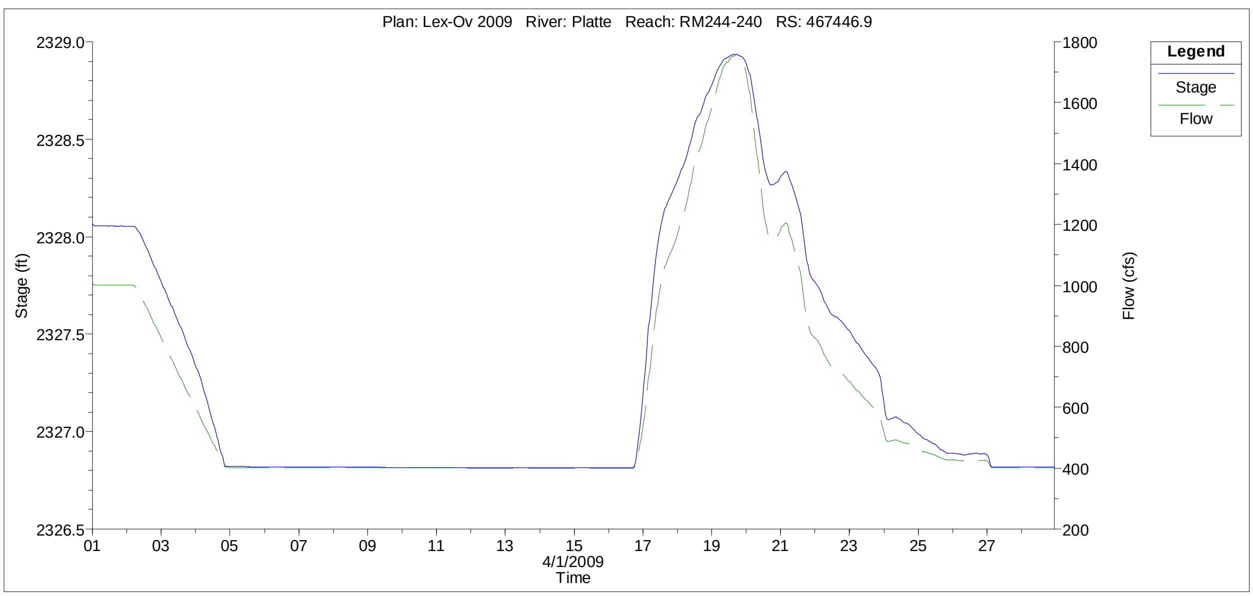


Figure 2. April 1998 hydrograph immediately upstream of the flow diversion return.

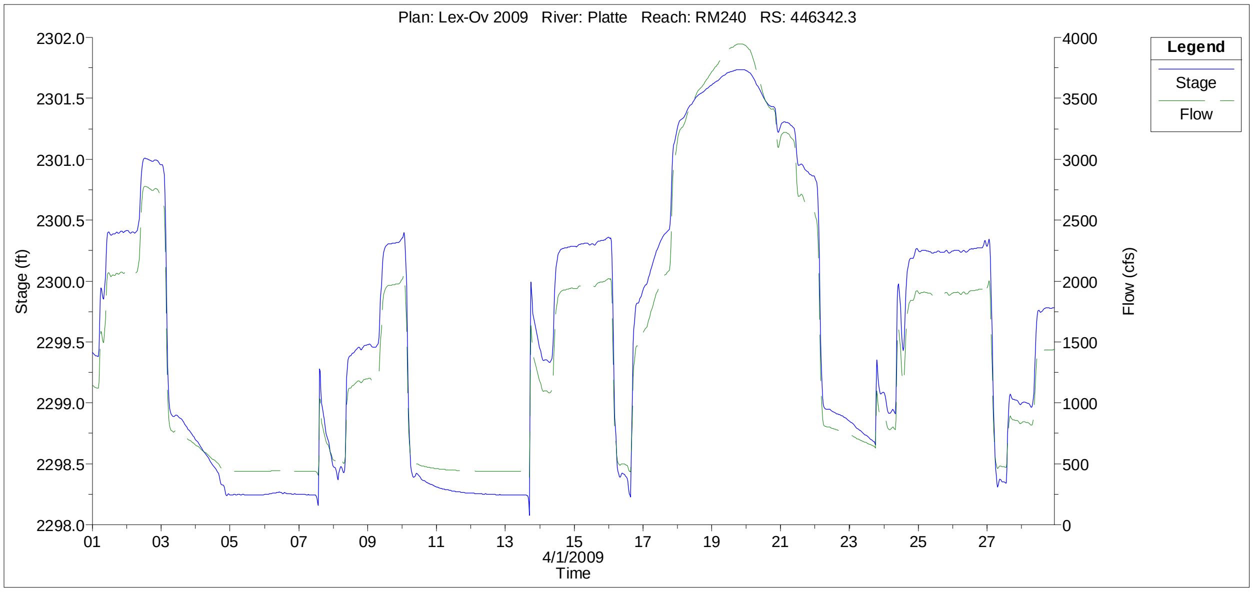


Figure 3. Resulting hydrograph after combining flows from upstream at North Platte and the J2 diversion.

Despite the lack of hydrograph consistency between North Platte and Overton, the hydrographs at Overton (after the Johnson 2 inflow) and Kearney (river mile 215) could be compared since there are no major diversions or inflows within this reach. Attenuation between these two locations is relatively small compared with peak flow rates. For example, routing the April 2009 flood hydrograph through this reach resulted in the peak discharge time lag of 7.5 hours between Overton and Kearney. The peak flow at each location was reduced by only 2.5% (3946 cfs to 3849 cfs). In addition, the peak flows at each location remained relatively constant over at least a 5.5 hour time interval. Thus, the assumption that flow is steady is appropriate since relatively little attenuation occurs and peak flows are fairly constant over relatively long time periods. Further verification that attenuation does not occur is provided by Cunge, Holly and Verwey (1980) who suggest that loops usually do not occur in rating curves for slopes steeper the average slope of 0.001. The slope between Overton and Kearney is 0.00118 ft/ft. Finally, the application of a kinematic wave model to approximate the flood hydrograph was estimated using a method provided by Ponce (1989). A kinematic wave is a simple translation of the inflow hydrograph downstream without any attenuation (Sturm 2001). To test this applicability, the following relationship must be met:

$$\frac{T_r V_0 S_0}{y_0} > 85$$

Where V_0, y_0, S_0 represent the average flow velocity, depth and slope, respectively. T_r represents the time of rise of the inflow hydrograph. Applying this relationship to the April, 2009 flood with an average velocity of 3.04 ft/s, an average depth of 4.79 ft, an average slope of 0.00118, and hydrograph rise time of 1.38 days, results in a value of 89. Moreover, the peak flow event in April, 1998 (peak flow rate ~5,600 cfs) resulted in a value of 129. These results confirm that the kinematic wave approximation is valid. The kinematic wave is essentially simulating a steady-state flow at each time step. The resulting hydrograph between the Tri-County Diversion and Lexington also shows this simple translation of the flood wave (Figure 5).

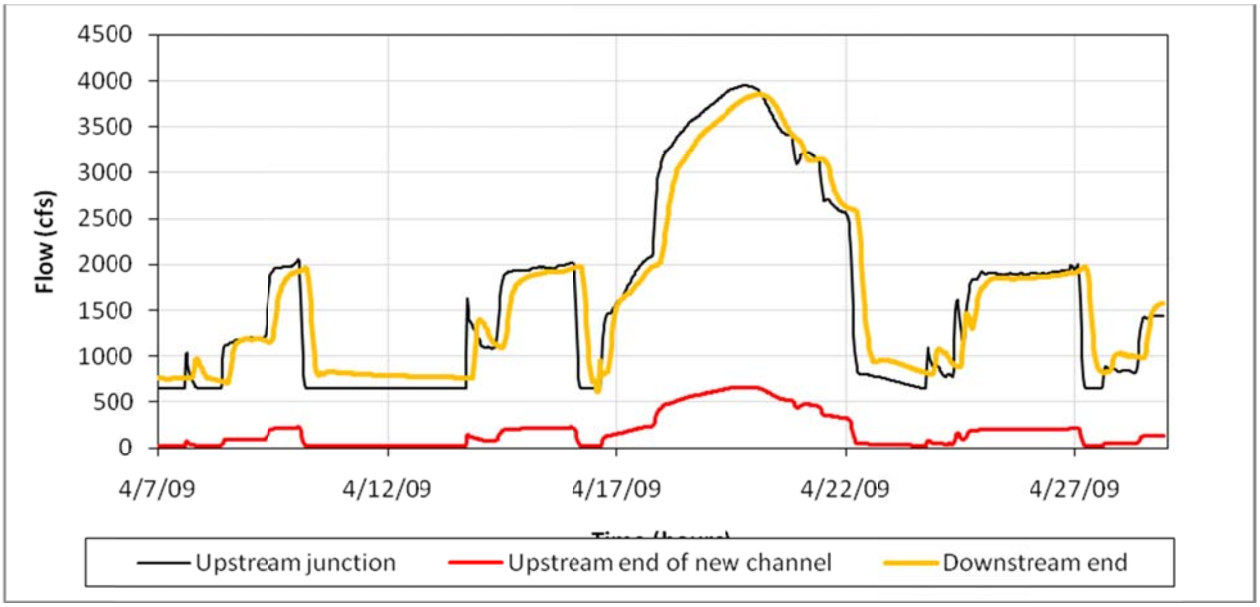


Figure 4. Comparison of hydrographs of the existing Platte River model at Overton, Lexington, and just upstream of overflow channel location at Cottonwood Ranch

Table 1. Unsteady flow characteristics around the peak flow rate during the April, 2009 flood.

Date, time	***Stage***	***Discharge (cfs)***
4/19/09 17:00	2301.31	3940.21
4/19/09 17:30	2301.32	3942.58
4/19/09 18:00	2301.32	3944.17
4/19/09 18:30	2301.32	3945.35
4/19/09 19:00	2301.32	3945.81
4/19/09 19:30	2301.32	3946.04
4/19/09 20:00	2301.32	3945.73
4/19/09 20:30	2301.32	3945.08
4/19/09 21:00	2301.32	3944.36
4/19/09 21:30	2301.32	3943.46
4/19/09 22:00	2301.32	3942.3
4/19/09 22:30	2301.31	3940.51

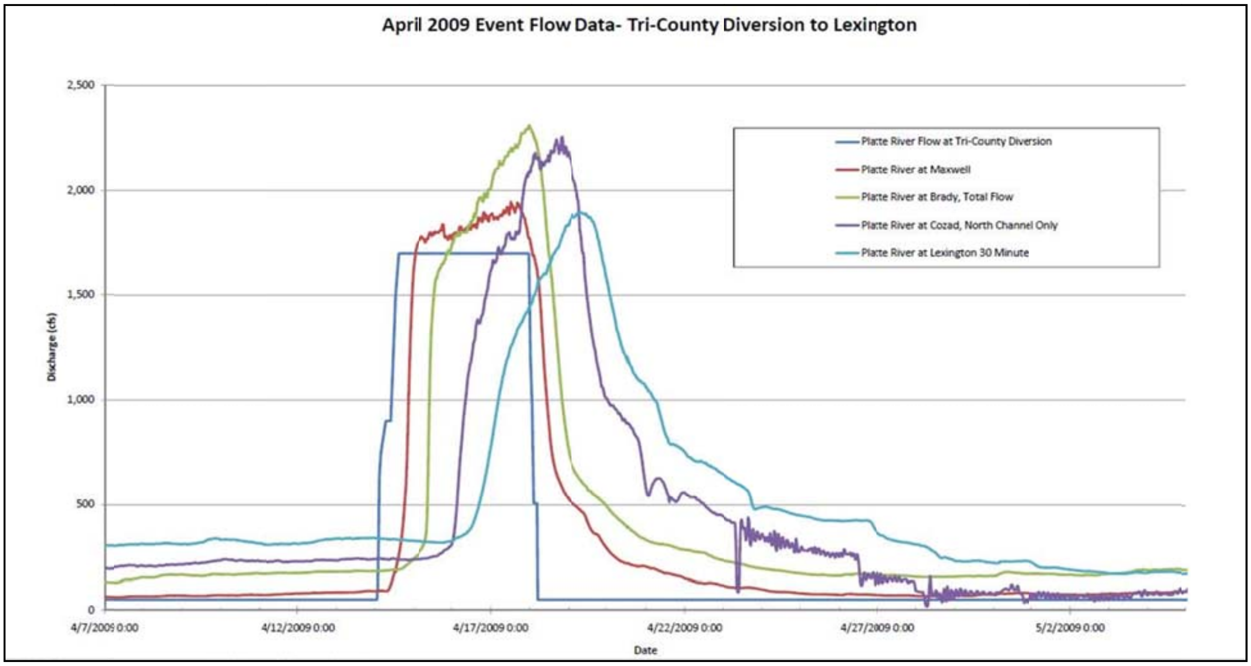


Figure 5. Comparison of hydrographs upstream from the major regulated flow input (J2).

Steady and unsteady flow accounting

Discharges in each reach did not match between the unsteady flow model with a constant flow rate, and the same flow in the steady-state model. This occurred because some connecting reaches between side channels and the main channel were deleted to stabilize the unsteady model. For example, a minimum of 500 cfs was required for stability in model runs. With the multiple connecting channels, small flow rates led to some channels either becoming dry or having supercritical flows which caused instability. To circumvent this problem, these connecting channels were combined into one channel with divided flow (Figure 6). This introduced some potential problems since HEC-RAS requires the water surface in each of the divided flow areas within a cross section to be equal. It is not likely that the errors introduced with this assumption are significant; however, a simple accounting for discharges in each channel revealed that differences occurred between the steady and unsteady models for the same total flow rate. Combining flows from two channels into one reach does not allow the deleted reach to transfer flow between reaches (e.g., from the South channel to the North channel).

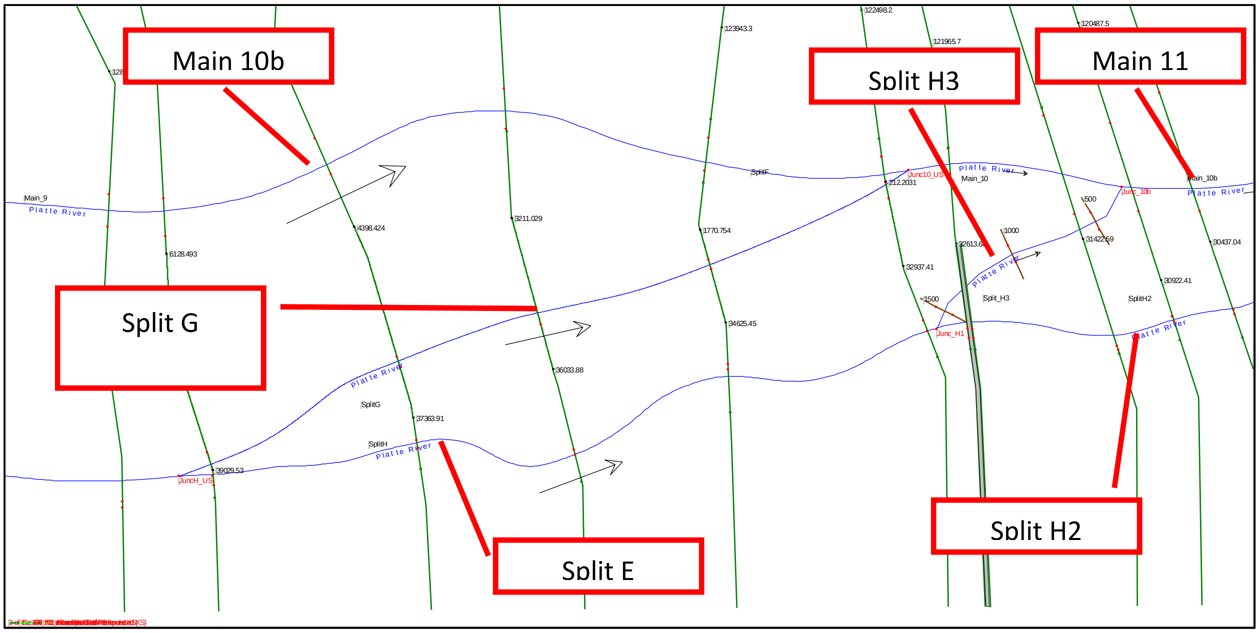


Figure 6. Plan view of the reaches modeled in the steady-state flow model. For the unsteady flow model, Split G was deleted and the cross sections in Split E were extended to include this flow area.

For instance, Split G was deleted in the unsteady flow model and combined with Split E (Figure 6). Divided flow conditions resulted in Split E, and flow was no longer able to move from the south channel to the north main channel in this location. Instead, most of this flow was pushed into the new Split H3. Routing a constant 8000cfs in the unsteady model, flow in Split H3 increased to 1602.27 cfs from 1340.49 cfs. This additional 261.78 cfs approaches the 316.31 cfs that was found in Split G with the steady-state model, though some of this additional water flows through Split H3. Increasing flows by 20% was unacceptable to understand the hydraulics in the reaches associated with the new overflow channel and hydraulic control.

Table 2. Comparison of steady-state model and the unsteady flow model using a constant discharge.

Reach	***Steady flow (cfs)***	***Unsteady flow (cfs)***
Split E (upstream from Split G)	1985.29	1868.67
Split E (downstream from Split G)	1669.08	1868.67
Split G	316.31	0 (Combined with Split E)
Split H2	328.59	266.40
Split H3	1340.49	1602.27
Main 10b	6922.88	7087.48
Main 11	6738.22	7087.45

CONCLUSIONS

Steady-state modeling was found to be appropriate for analyzing hydraulic characteristics in the region of the Cottonwood Ranch project. Hydrograph comparisons between different river stations verified that little attenuation occurs, and flood peaks do not rise or fall so fast that a steady-state model is not applicable. Suggested slopes by Cunge, Holly and Verwey (1980), and an empirical relationship provided by Ponce (1989) corroborate this result. In addition, it was discovered that simplification of the geometry in the steady-state model to allow stability in the unsteady model introduced unacceptable errors. Deleted reaches in the unsteady model introduced additional flow into the newly designed reach, misrepresenting actual flow processes. As a result, the steady-state model was used for the design of the Cottonwood Ranch log jam.

REFERENCES

Cunge, J.A., F.M. Holly Jr., and A. Verwey. 1980. *Practical aspects of computational river hydraulics*. Pitman. London

HDR Engineering Inc, Tetra Tech Inc., and The Flatwater Group Inc. 2011. *1-D Hydraulic and Sediment Transport Model: Final Hydraulic Modeling Technical Memorandum*. Prepared for: Platte River Recovery Implementation Program.

Ponce, V.M. 1989 *Engineering Hydrology: Principles and Practice*. Prentice-Hall. Englewood Cliffs, New Jersey.

Sturm, T.W. 2001. *Open channel hydraulics*. McGraw-Hill. Boston, Massachusetts.

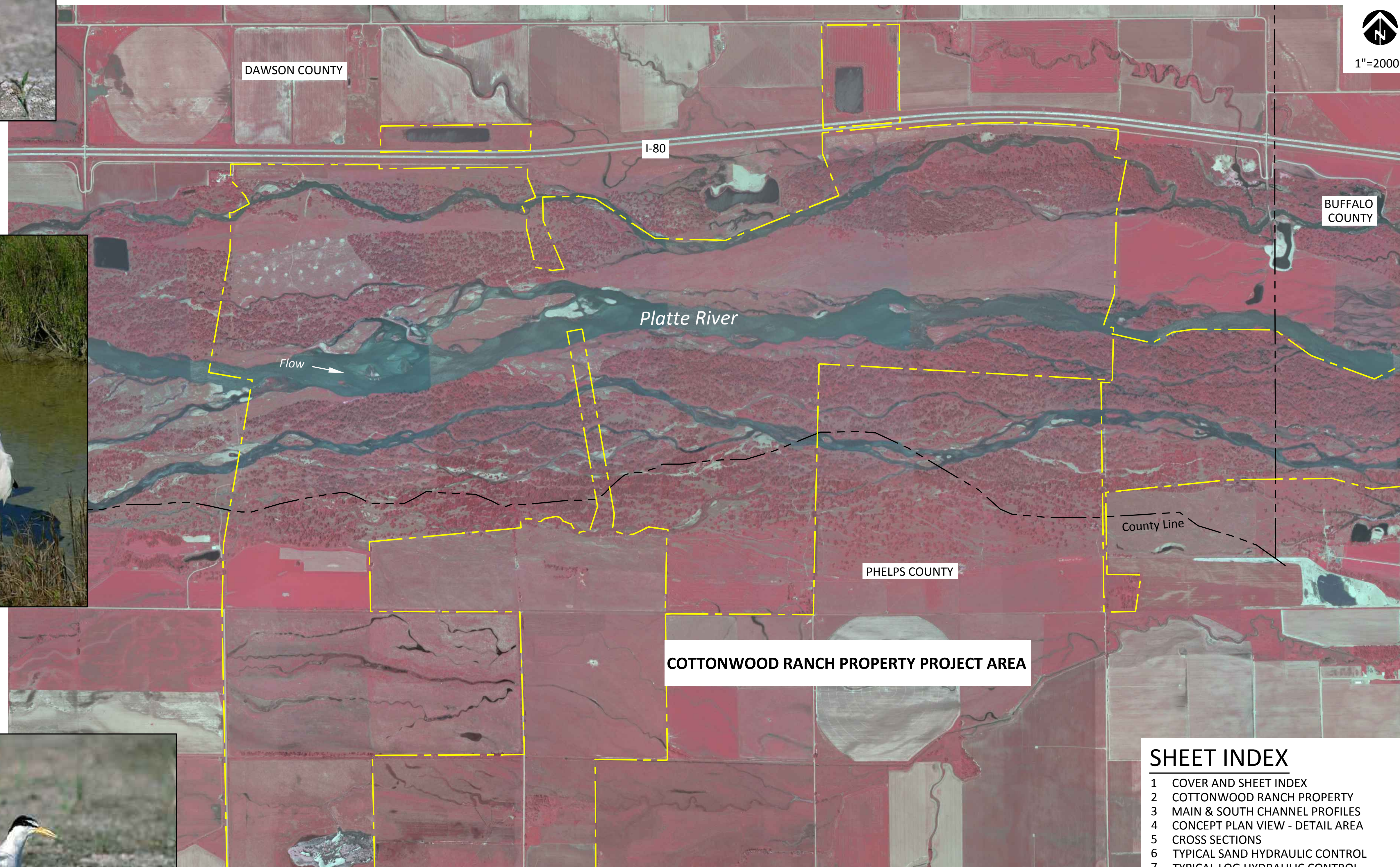
Flow Consolidation Methods for Habitat Recovery

Platte River, Nebraska

PIPING PLOVER
Charadrius melodus

WHOOPING CRANE
Grus americana

LEAST TERN
Sterna antillarum



SITE MAP

SHEET INDEX

1 COVER AND SHEET INDEX
2 COTTONWOOD RANCH PROPERTY
3 MAIN & SOUTH CHANNEL PROFILES
4 CONCEPT PLAN VIEW - DETAIL AREA
5 CROSS SECTIONS
6 TYPICAL SAND HYDRAULIC CONTROL
7 TYPICAL LOG HYDRAULIC CONTROL

PREPARED FOR:
PLATTE RIVER RECOVERY IMPLEMENTATION PROGRAM,
'THE PROGRAM'
KEARNEY, NEBRASKA
MAY 12, 2011

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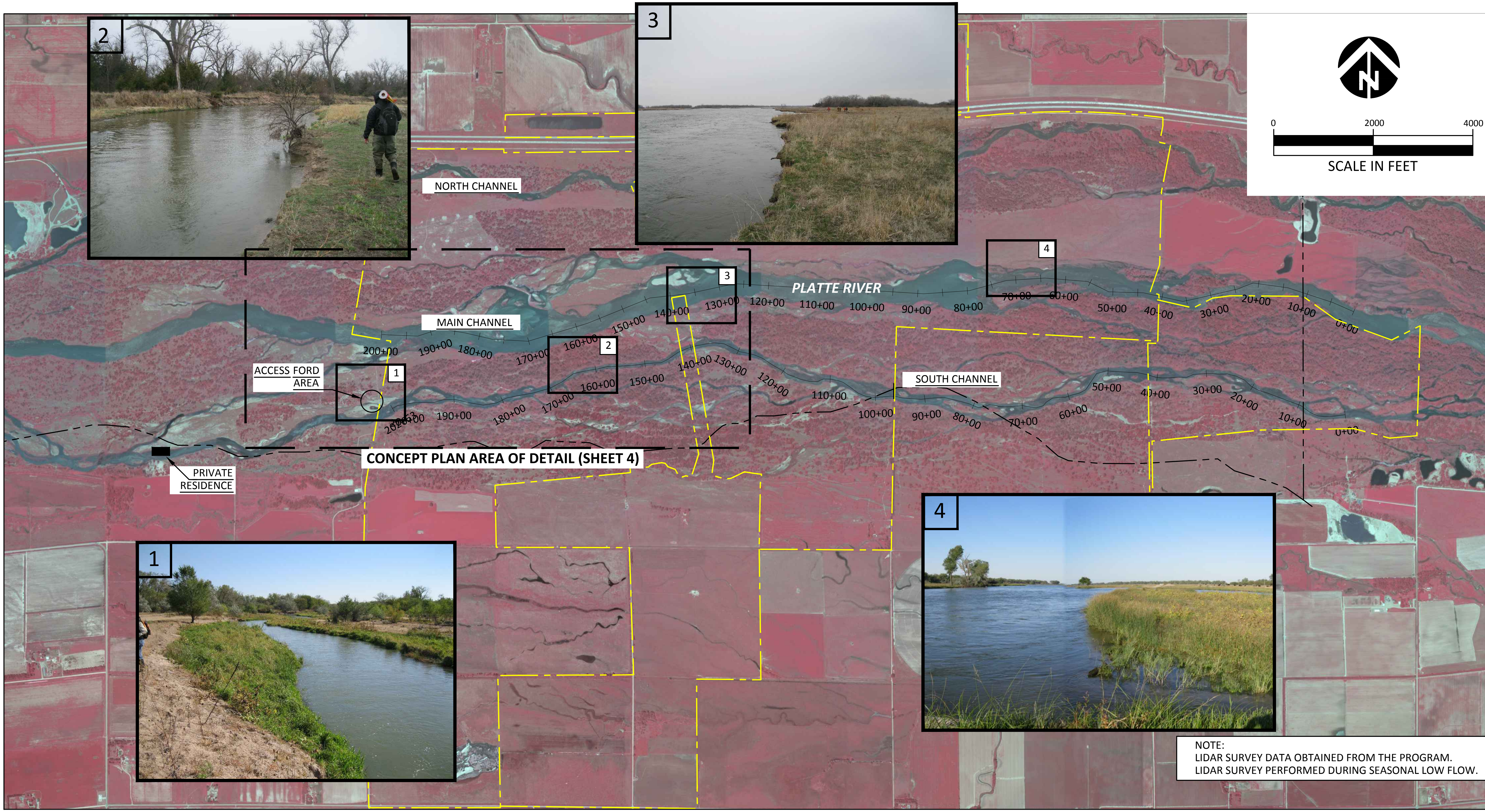
Cover Sheet and Index



PLAN OVERVIEW

LEGEND

COTTONWOOD RANCH PROPERTY BOUNDARY

2+00 CHANNEL CENTERLINE

COUNTY LINE

NOTE:
COLOR INFRARED IMAGE OBTAINED FROM 'THE PROGRAM'. DATE OF IMAGING - JUNE 17, 2010.

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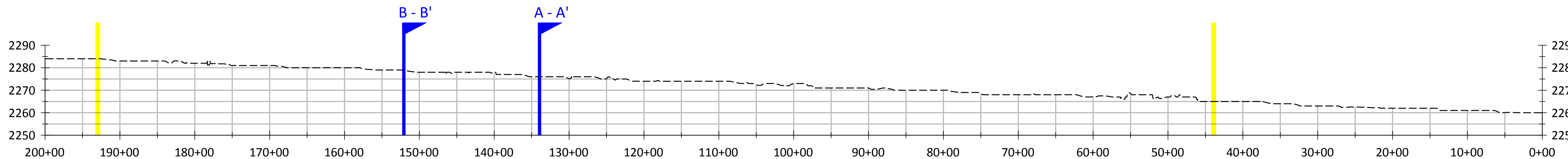
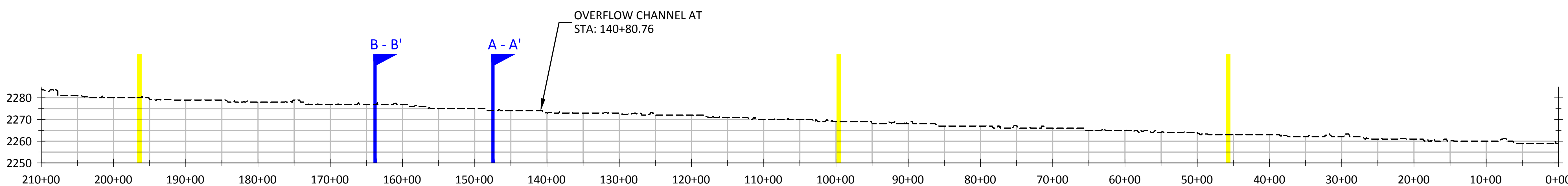
Cottonwood Ranch Property

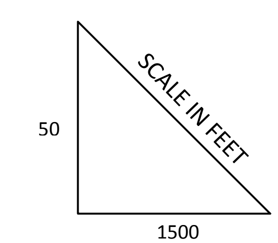


MAIN CHANNEL PROFILE
PLATTE RIVER



SOUTH CHANNEL PROFILE
PLATTE RIVER

NOTE:
LIDAR SURVEY DATA OBTAINED FROM THE PROGRAM.
LIDAR SURVEY PERFORMED DURING SEASONAL LOW FLOW.



LEGEND

APPROXIMATE LOCATION WHERE CHANNEL FLOWLINES CROSS THE COTTONWOOD RANCH PROPERTY BOUNDARY

EXISTING BANK ELEVATION AT CHANNEL WATER SURFACE AS OBTAINED FROM LIDAR SURVEY DATA

CROSS SECTION LOCATION WITH RELATIONSHIP TO CHANNELS - SEE SHEETS 4 AND 5

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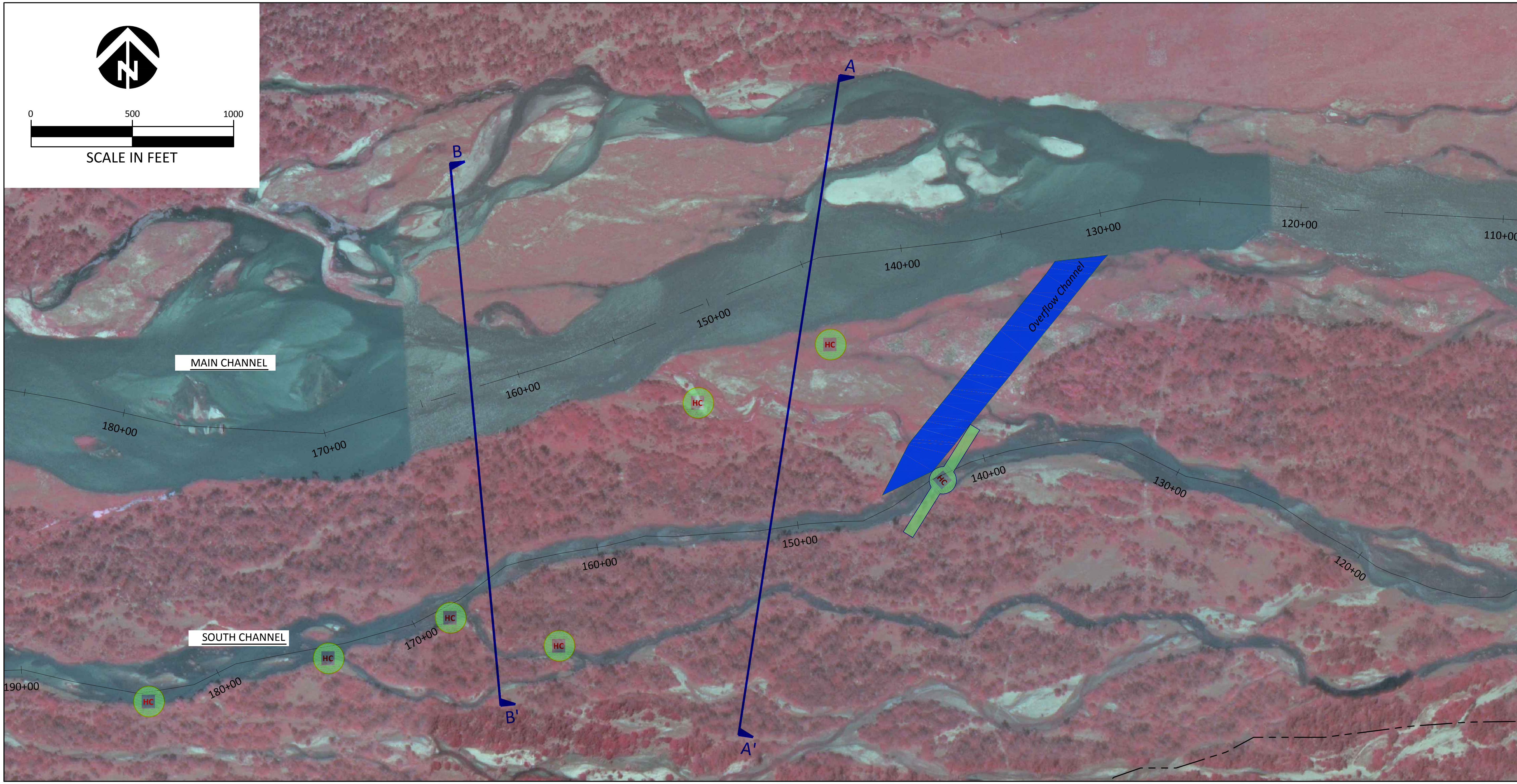
Main & South Channel Profiles



CONCEPT PLAN VIEW

LEGEND

2+00

CHANNEL FLOWLINE

HC MINOR HYDRAULIC CONTROL (SEE SHEET 6)

HC MAJOR HYDRAULIC CONTROL (SEE SHEET 6)

NOTE:
COLOR INFRARED IMAGE OBTAINED FROM 'THE PROGRAM'. DATE OF IMAGING - JUNE 17, 2010. DEPICTED WATER INUNDATION ON IMAGING DATE REPRESENTS A FLOW OF 8,000 cfs.

NOTES:
MAJOR HYDRAULIC CONTROLS ARE INTENDED TO BE SUBSTANTIAL STRUCTURES OCCUPYING THE SOUTH CHANNEL AND SPANING WELL INTO THE FLOODPLAIN. THEY MAY BE EITHER LOG JAMS OR SIGNIFICANT SAND PLUGS.

ALTHOUGH NOT SHOWN, MAJOR AND MINOR HYDRAULIC CONTROLS MAY BE DUPLICATED THROUGHOUT THE DOWNSTREAM REACH OF THE SOUTH CHANNEL AS NECESSARY.

MINOR HYDRAULIC CONTROLS ARE INTENDED TO BE VEGETATED SAND PLUGS OR SMALL LOG JAMS TO SEAL OFF ADJACENT FLOWPATHS COMING OFF THE SOUTH OR MAIN CHANNEL.

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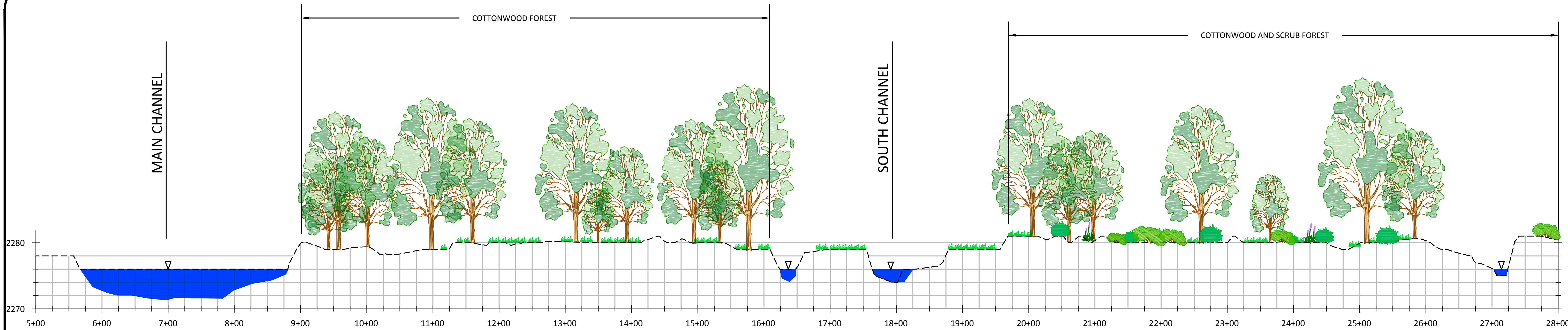
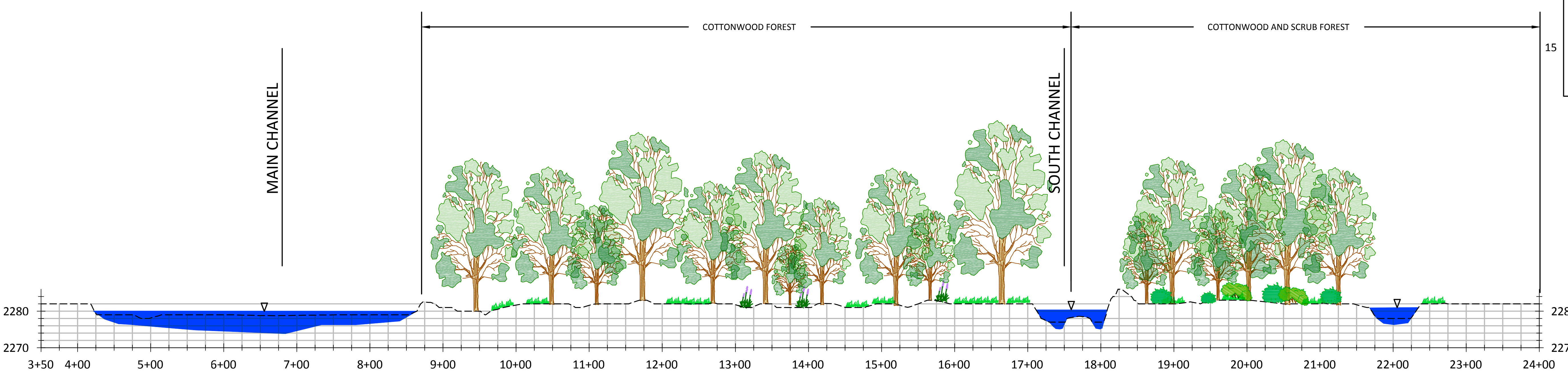
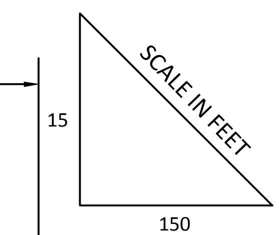
Concept Plan Area of Detail



CROSS SECTION A-A'





CROSS SECTION B-B'

NOTE:
EXISTING GROUND SURFACE OBTAINED FROM LIDAR SURVEY DATA. THIS DATA DOES NOT INCLUDE INFORMATION ON GROUND ELEVATION BELOW THE WATER SURFACE. DEPTH OF WATER SHOWN IS ILLUSTRATIVE ONLY.

LEGEND

CHANNEL (DEPTH SHOWN IS ILLUSTRATIVE)

EXISTING GROUND AND WATER SURFACE AS OBTAINED FROM LIDAR SURVEY DATA

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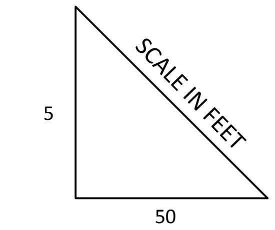
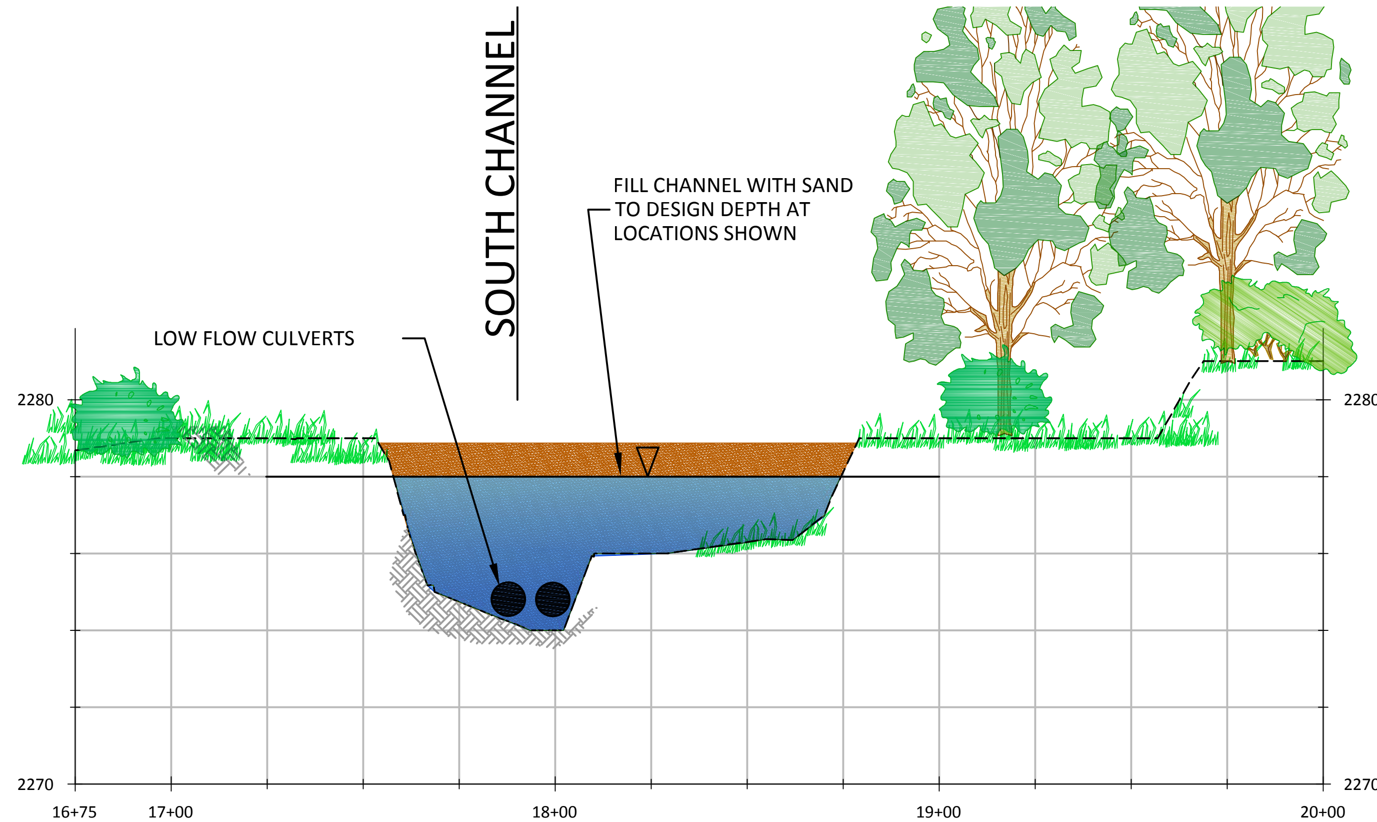
Existing Conditions Cross Sections



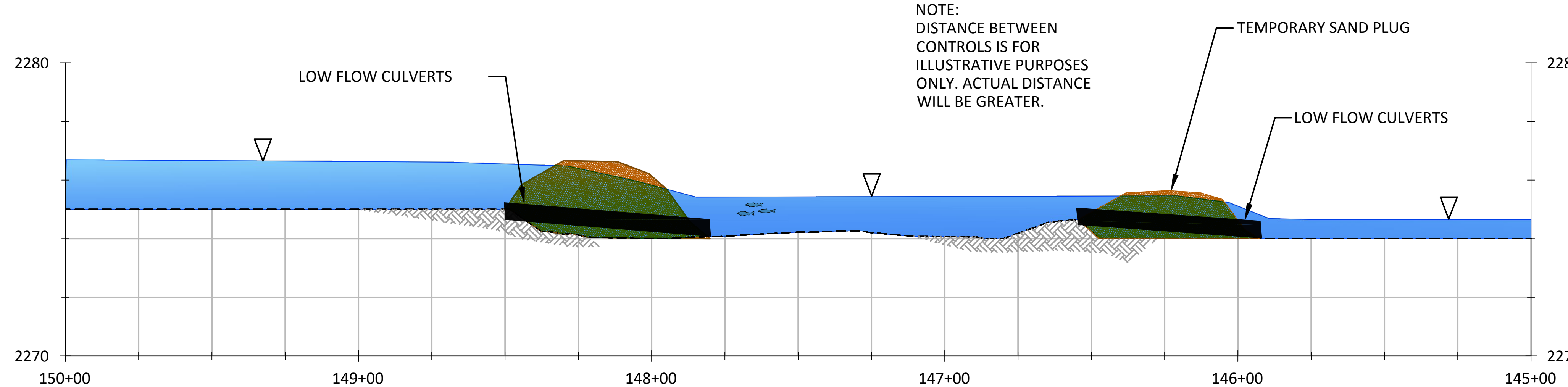
NOTE:

REFERING TO SHEET 4, THE DIFFERENCE BETWEEN MINOR AND MAJOR HYDRAULIC CONTROLS IS SIMPLY THE SCALE OF EITHER THE SAND OR LOG STRUCTURE.



ELEVATION

SAND PLUG
(Temporary Control)



PROFILE

SAND PLUG
(Temporary Control)

LEGEND

CHANNEL FLOW (DEPTH SHOWN IS ILLUSTRATIVE)

SAND

EXISTING GROUND AND WATER SURFACE AS OBTAINED FROM LIDAR SURVEY DATA

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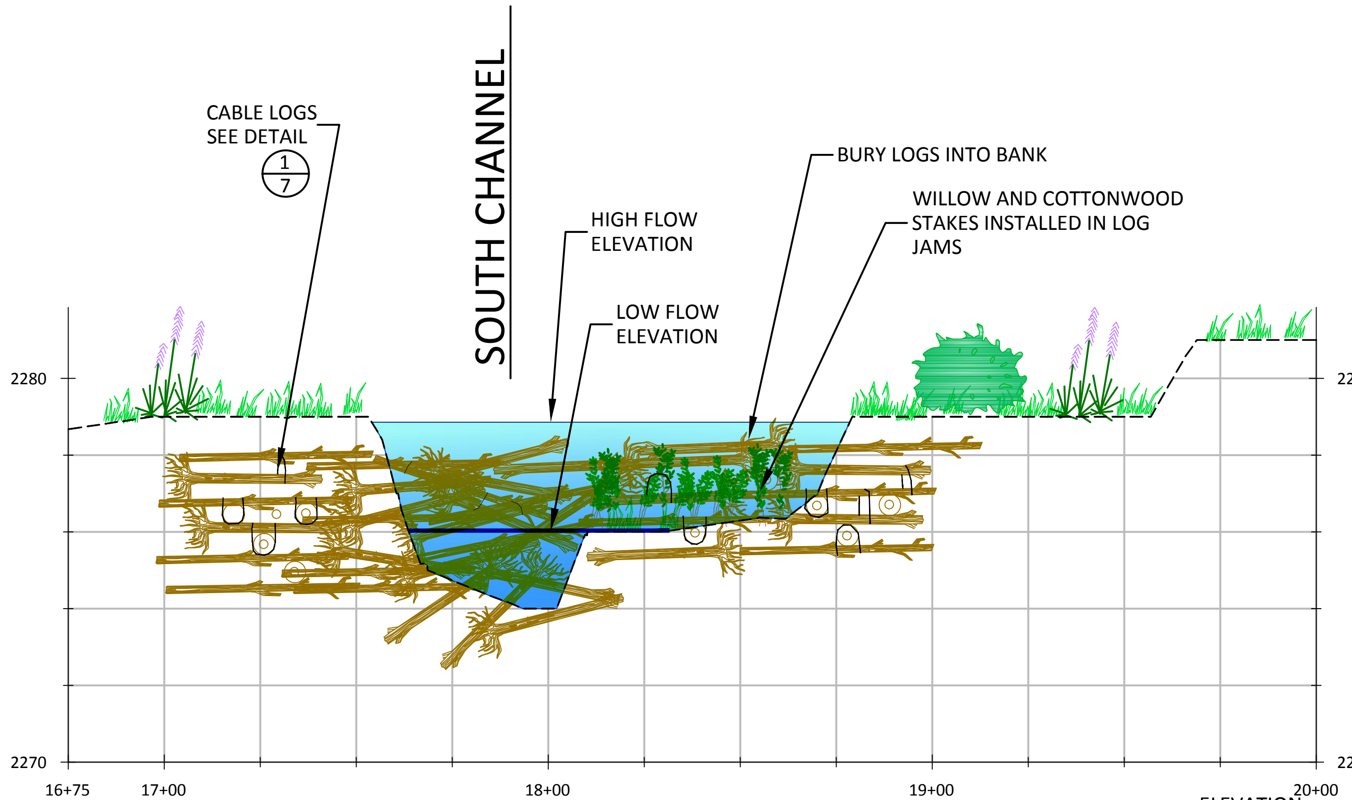
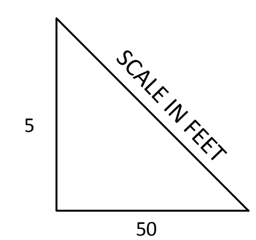
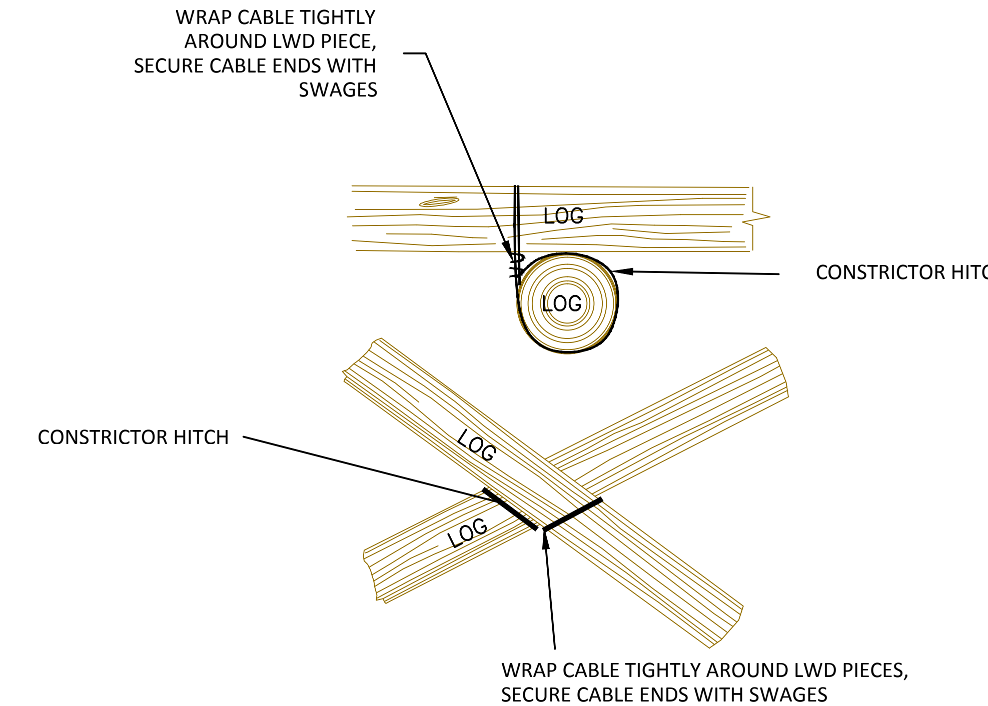
TYPICAL SAND HYDRAULIC CONTROL

SOUTH CHANNEL



LOG JAM
(Permanent Control)



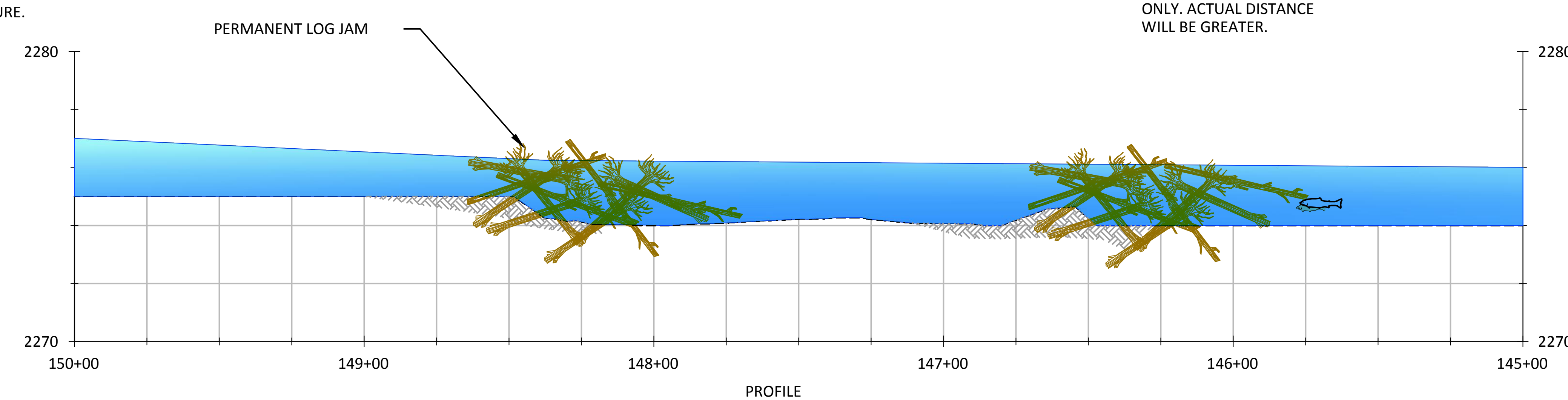


1/7 DETAIL VIEWS
TYPICAL LOG CABLING
NOT TO SCALE

NOTE:

REFERING TO SHEET 4, THE DIFFERENCE BETWEEN MINOR AND MAJOR HYDRAULIC CONTROLS IS SIMPLY THE SCALE OF EITHER THE SAND OR LOG STRUCTURE.

NOTE:
DISTANCE BETWEEN CONTROLS IS FOR ILLUSTRATIVE PURPOSES ONLY. ACTUAL DISTANCE WILL BE GREATER.



LOG JAM
(Permanent Control)

LEGEND

CHANNEL FLOW (DEPTH SHOWN IS ILLUSTRATIVE)

EXISTING GROUND AND WATER SURFACE AS OBTAINED FROM LIDAR SURVEY DATA

LOCAL COTTONWOOD LOGS

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Typical Log Hydraulic Control