A&B Irrigation District Ground Water-to-Surface Water Conversion Project Study

Submitted to

Idaho Water Resource Board

July 2008

MWH

A&B Irrigation District
Ground Water-to-Surface Water Conversion Project Study

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Submitted by

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EXECUTIVE SUMMARY

The Eastern Snake Plain Aquifer Comprehensive Aquifer Management Plan Advisory Committee has determined that the conversion of Unit B of the A&B Irrigation District from groundwater to surface water will have a positive effect on the ESPA water budget by reducing groundwater withdrawals. The following study evaluates two conceptual alternatives to deliver surface water to Unit B: 1) gravity diversion from the Snake River and 2) pressurized diversion from the Snake River. In addition, delivering surface water to Unit B for groundwater recharge was also evaluated as a third alternative.

For the purposes of this study, it was assumed that a surface water supply will be available from the Snake River to completely replace groundwater pumping in Unit B. After evaluation of historical groundwater pumping records from 1995-2007 and a review of U.S. Bureau of Reclamation project design data, it was determined that the surface water distribution system would be conceptually designed with a capacity of 1,000 cubic feet per second.

The gravity diversion alternative was envisioned to provide Unit B with surface water at a lower operational cost. It was determined that the nearest gravity diversion from the Snake River needed to be located just downstream of Idaho Falls. Constructing the required 100-mile pipeline would be cost prohibitive and was eliminated from further consideration.

Numerous variations of the pressurized diversion alternative were considered and compared, including single and multiple diversion points and main pipelines. An alternative with a single diversion point off of Lake Walcott and two main pipelines providing service to Unit B appeared to be the most feasible and economical at a conceptual level.

Groundwater recharge options were considered to determine the feasibility of recharging the groundwater within Unit B to allow the continued use of existing groundwater wells. Review of existing data sources indicated that the geology within Unit B was not ideal for either shallow infiltration basins or vadose zone injection wells. Required treatment of water prior to injection would make aquifer injection wells and vadose zone injection wells cost prohibitive.

After conceptual evaluation of the three alternatives, the pressurized diversion alternative with a single diversion point and two main pipelines was selected as the recommended conceptual alternative. Refer to Figures 6 and 7 for a conceptual layout of the distribution system. An Association for the Advancement of Cost Engineering (AACE) International Class 5 cost estimate of the alternative yields an opinion of probable project cost of $360,000,000 and estimated annual operational costs of $3,400,000. As defined by the AACE, International Class 5 cost estimates have a maximum accuracy range of -50 percent on the low side and +100 percent on the high side.

Although the opinion of probable project cost of the recommended conceptual alternative is high, MWH recommends that additional study be conducted based on the available water supply that is identified and quantified. As noted in the study, the main pipelines of the distribution system constitute the largest portion of the project cost and any reduction in system capacity will have a large impact on the overall cost of the system. Therefore, it is imperative that the system is sized

to a capacity that optimizes the timing and magnitude of the actual water supply to be delivered. In addition, strategically located storage facilities could offset the required size of the main pipelines as well, and should be more comprehensively evaluated in a future study.

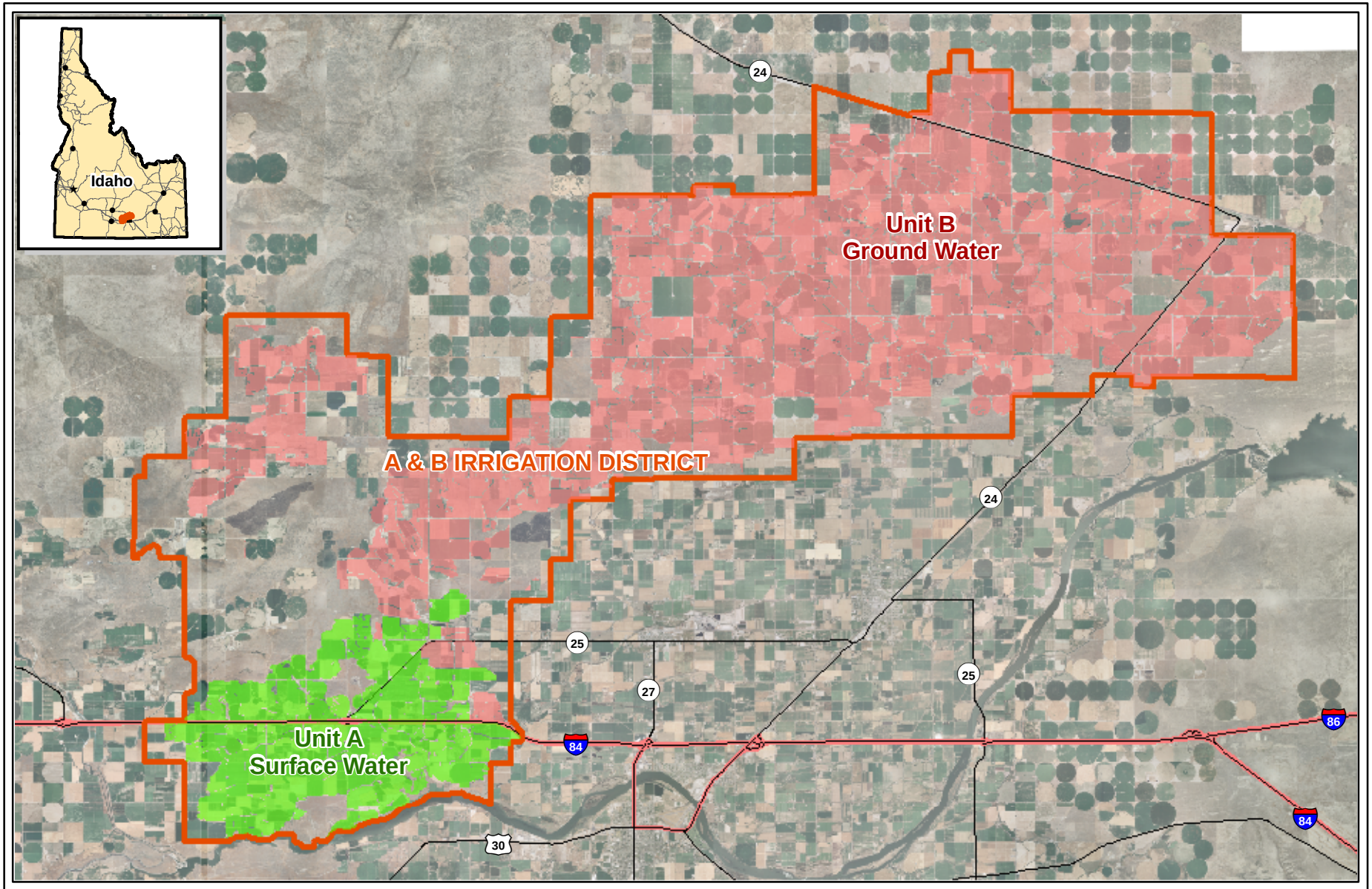
If the amount of available water supply is inadequate to meet the total irrigation demand of Unit B, a conjunctive use operation could be implemented where surface water is supplied when available and groundwater is supplied when surface water is unavailable. A&B Irrigation District may be able to realize operational efficiencies with this type of a program and the availability of a surface water distribution system may allow access to surplus water in the non-irrigation season for storage in reservoirs or in the aquifer if further field exploration determines that groundwater recharge is a viable option.

MWH also recommends that the feasibility of providing partial surface water replacement to Unit B be considered. This alternative would eliminate a portion of the groundwater withdrawals by providing surface water to the northern part of Unit B and has the potential to provide incidental recharge from the surface water irrigation deliveries upgradient of a majority of the remaining groundwater wells. In addition, less surface water supply would be required and the capital cost of the surface water distribution system could be substantially less than the recommended conceptual alternative opinion of probable project cost presented in this study. Determining the incidental recharge benefit of the irrigation deliveries will be necessary to evaluate the net effect of this alternative on the Eastern Snake Plain Aquifer. If this option is considered, it is recommended that exploratory drilling and detailed soil analysis be conducted in the northeast part of Unit B.

INTRODUCTION

The A&B Irrigation District (A&B) is situated in southern-central Idaho near the communities of Burley and Rupert. As the name suggests, A&B is divided into two areas, Unit A and Unit B (Figure 1). Unit A is comprised of approximately 16,000 acres of the southwest portion of A&B immediately adjacent to the Snake River. Surface water is diverted for Unit A directly from the Snake River approximately seven miles west of Burley. Unit B is comprised of approximately 67,000 acres stretching northeast from the northern edge of Unit A. A network of groundwater wells provides irrigation water for Unit B (Figure 2).

The Eastern Snake Plain Aquifer (ESPA) Comprehensive Aquifer Management Plan Advisory Committee has determined that the conversion of Unit B from groundwater to surface water will have a positive effect on the ESPA water budget by reducing groundwater withdrawals. This study provides a conceptual analysis of distribution system alternatives to supply Unit B with surface water, with the intent to select a recommended conceptual alternative and provide a preliminary opinion of the probable project and operational costs to enable the Idaho Water Resource Board (IWRB) to assess the merit of the proposed project and determine if further analysis and design is warranted. For the purpose of this study, it is assumed that additional surface water will be available from the Snake River to completely replace groundwater pumping by A&B.



Legend

- Unit A Surface Water Irrigation Area
- Unit B Ground Water Irrigation Area
- Irrigation District
- Interstate
- US & State Hwys

A & B Irrigation District

Units A & B

0 1.5 3 6 9 Miles

Figure 1

Date: June-2008

Spatial Reference: IDTM, NAD-83

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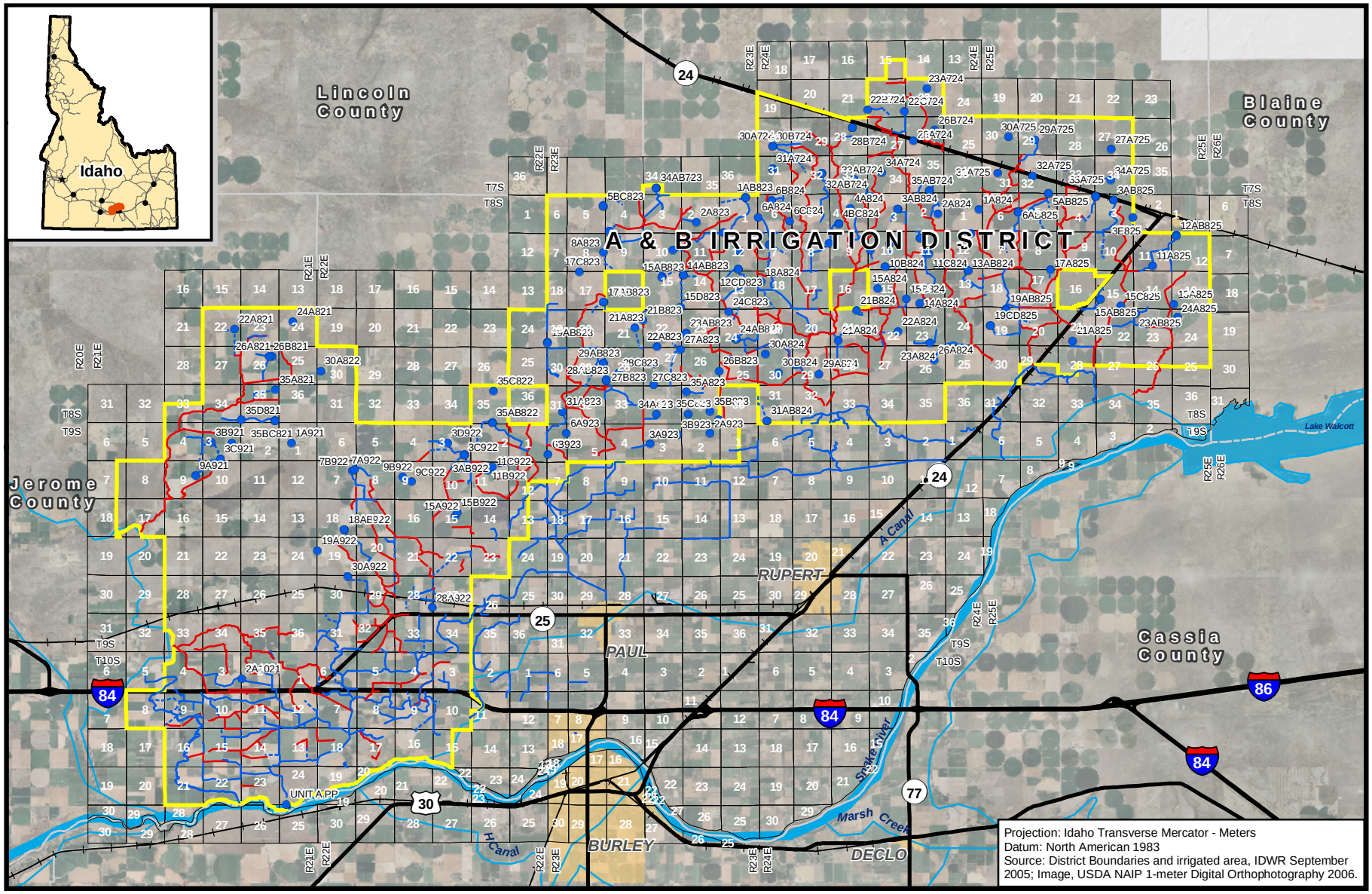

Projection: Idaho Transverse Mercator - Meters
Datum: North American 1983
Source: District Boundaries and irrigated area, IDWR September 2005; Image, USDA NAIP 1-meter Digital Orthophotography 2006.

Legend

- Active Well
- Drain
- Lateral
- Pipeline
- A & B ID Boundary
- Section Boundary
- Highways
- Railroad
- Canal
- Snake River
- Municipality
- Water Body
- County Boundary

A & B Irrigation District
Current Conveyance System and Active Wells

0 1 2 4 6 8 Miles

Figure 2

Date: June-2008

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CONCEPTUAL ALTERNATIVE DEVELOPMENT AND EVALUATION

GENERAL ASSUMPTIONS

In addition to the overarching assumption that an adequate surface water supply is available from the Snake River, the following assumptions were made to assist in evaluation of the conceptual alternatives where appropriate.

Distribution System Capacity

Unit B groundwater records between 1995 and 2007 were provided to MWH by the IWRB staff. The records were compiled to determine the maximum monthly volume used by A&B since 1995 (Appendix A). This monthly volume was converted to a flow rate of 900 cubic feet per second (cfs). After review of project summary information from the U.S. Bureau of Reclamation (Reclamation), it was discovered that the Unit B groundwater wells were designed to deliver approximately 1,000 cfs (Reclamation 1961). It was decided that the conceptual distribution system alternatives should be sized to accommodate 1,000 cfs, which would match the design capacity of the groundwater wells and provide a 10 percent peaking factor on the flow rate calculated from the historical maximum monthly volume.

No attempt was made to evaluate the irrigation efficiency on lands within A&B. Depending on current application methods being used within A&B, it is possible that an increase in irrigation efficiency may reduce the required capacity of the surface water distribution system.

Distribution System Criteria

The proposed distribution system laterals will deliver water to and connect directly into the existing groundwater delivery systems for subsequent delivery to individual landowners.

No attempt was made to evaluate the distribution efficiency of the existing groundwater delivery systems, although it was noted during the site visit (Refer to Site Visit Report – Appendix B) that many systems are comprised of unlined open channels which may provide an opportunity to increase efficiency and reduce the required capacity of the surface water distribution system. However, it should be noted that increased efficiency may come at the expense of reduced groundwater recharge.

CONCEPTUAL ALTERNATIVES

Two alternatives for supplying surface water to Unit B, each with numerous variations, were initially evaluated: 1) gravity diversion from the Snake River and 2) pressurized diversion from the Snake River. In addition, supplying surface water to Unit B for groundwater recharge was evaluated as a third alternative. Each alternative was independently evaluated at a conceptual level both qualitatively and quantitatively to determine which variation of the alternative, or option, would likely be the most promising in terms of its delivery and cost effectiveness. The recommended conceptual alternative was then identified from the best apparent options from

each alternative and a Class 5 opinion of probable project cost was then prepared for that alternative.

Conceptual Alternative 1 – Gravity Diversion

The first conceptual alternative involves determining the nearest feasible gravity diversion point along the Snake River and the required conveyance system to deliver water to the northeast corner of Unit B. As stated in the scope of work, the distribution system required to deliver water to growers within A&B will not be part of this alternative. It is assumed that the gravity conveyance system can be sufficiently integrated into the distribution systems developed for Conceptual Alternatives 2 and 3 for comparative purposes at this preliminary level of study.

Diversion Location

While the formal evaluation of a diversion location should be based on topographic, engineering, environmental and water right criteria, the evaluation of the diversion location in this analysis was based entirely on topographic criteria. The eastern edge of Unit B is located at a ground elevation of approximately 4,300 feet. The diversion location needs to be at an elevation high enough above 4,300 feet to allow gravity flow to Unit B.

The major reservoirs immediately upstream of A&B, Lake Walcott Reservoir and American Falls Reservoir, are potential gravity diversion locations; however, the elevation difference and topography between the reservoirs and the eastern edge of Unit B will not permit gravity flow. Lake Walcott operates at an elevation of 4,245 feet; 55 feet below the elevation at the eastern edge of A&B. The maximum water surface elevation at American Falls Reservoir is approximately 50 feet higher than the eastern edge of A&B, but the topography rises away from the reservoir making a purely gravity system impossible. In addition, the Snake River enters a canyon below the dam that would make the construction of a conveyance facility very difficult. Therefore, it appears that both reservoir sites would require pumping water out of the reservoir to enable it to reach an elevation suitable for gravity flow.

With the elimination of the reservoir sites, the Snake River above American Falls Reservoir was considered. To locate the diversion point along the Snake River, the most appropriate conveyance method had to be determined. There are two main options that can be used to convey irrigation water to Unit B by gravity: an open channel system or a pipeline.

The main advantages of an open channel system are lower frictional head losses and capital costs; however, the disadvantages are higher water losses associated with seepage and evaporation and a longer, more sinuous, alignment because of the need to more closely follow the topographic contours. With water losses being of considerable concern, the open channel would likely require concrete lining which would negate much of the cost savings that would be expected with an unlined open channel. The additional required length also would reduce the cost savings and would affect a larger land area. In this regard, canals typically require the acquisition of large linear land tracts because the land is not useable for other purposes after the construction of the canal.

A pipeline would provide the lowest water losses by eliminating evaporation and seepage and would provide a less sinuous alignment by accommodating reasonable fluctuations in elevation. In addition, an operation and maintenance easement may be possible for a pipeline as opposed to land acquisition. However, a pipeline would require the highest elevation difference between the diversion location and eastern edge of A&B to overcome higher frictional losses, potentially requiring a more distant diversion point.

Ultimately, existing water supply conditions within the ESPA dictate that the most efficient conveyance method be used. In addition, from the discussion above it appears that the costs associated with each option appear to roughly balance at a conceptual level. Therefore, a pipeline was identified for further evaluation.

Pipeline Option

The location identified for the pipeline intake would be approximately 100 miles northeast of A&B on the Snake River near Idaho Falls. The pipeline would have a 16-foot inside diameter (I.D.) and the invert of the intake would be at an elevation of 4,650 feet (about 350 feet higher than the proposed discharge point at the eastern edge of Unit B). This is the approximate elevation difference required to overcome frictional losses within the 16-foot pipeline, including for additional losses from fittings and additional pipeline length required because of the vertical elevation fluctuations over the 100-mile alignment.

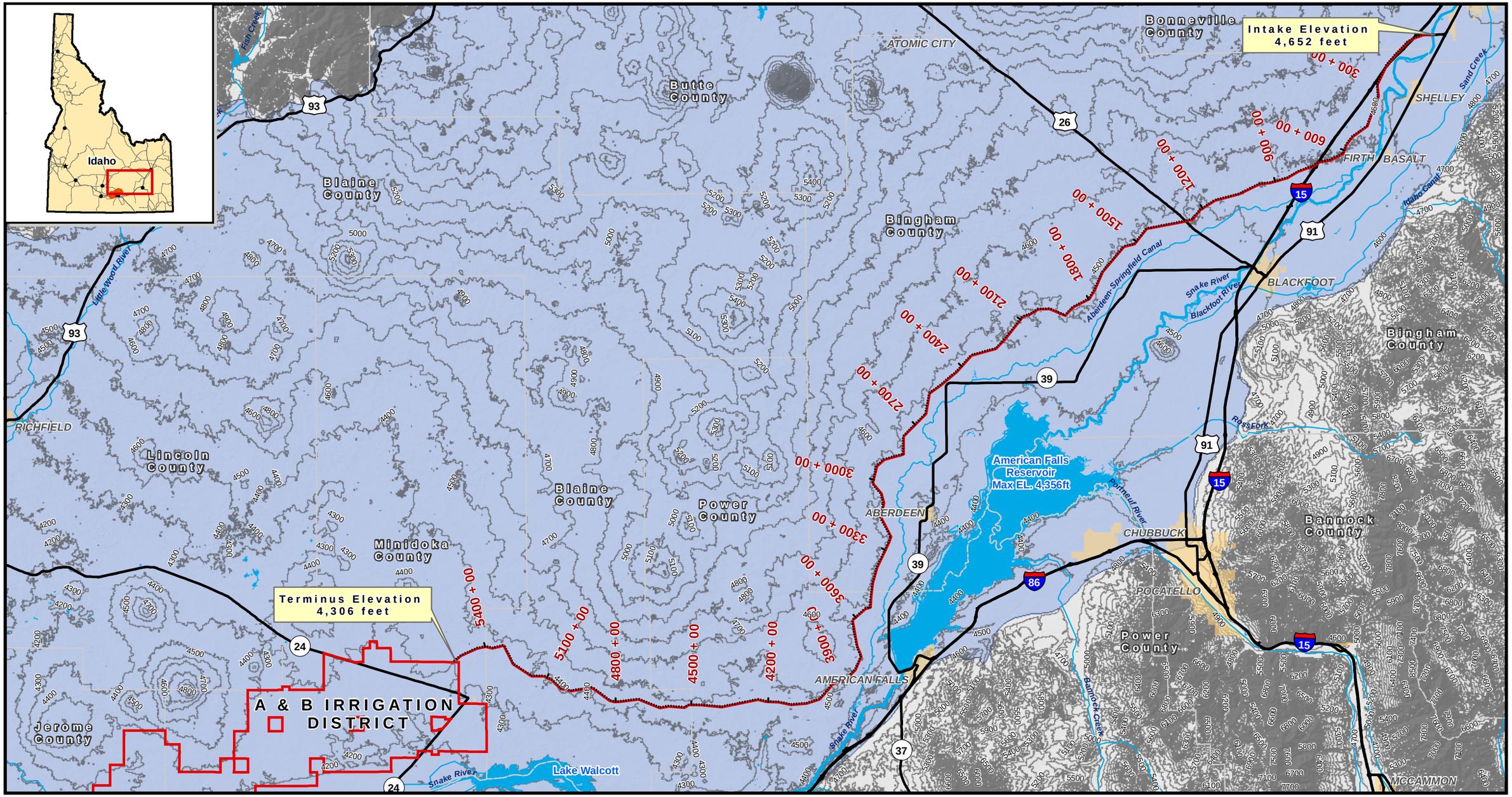
The pipeline would parallel the Aberdeen-Springfield canal on the high side down to American Falls Reservoir before extending in a westerly direction to reach A&B. Figure 3 shows the potential alignment of the A&B pipeline system.

Conclusions

As indicated below, the preliminary estimated cost of the gravity flow pipeline option would be extremely high, even without the cost of the distribution system within A&B.

Based on lineal foot installed unit costs, the construction of a 100-mile 16-foot I.D. buried pipeline (with associated fittings, valves, etc.) from Idaho Falls to the eastern edge of Unit B would cost more than $1.5 billion. This cost would not include the cost of the distribution system within A&B. In addition, the geology of the area suggests that the alignment would cross many areas with shallow basalt bedrock which presents construction difficulties.

Despite the immense effort and cost involved in conveying water by gravity to the eastern edge of A&B, it is doubtful that the distribution system within A&B can even be designed to operate by gravity alone due to the relatively small elevation difference across A&B. Therefore, Alternative 1 does not appear to be a suitable approach to provide irrigation water to A&B and is recommended for elimination from further consideration.



Legend

- Potential Gravity Diversion Pipeline
- A & B ID Boundary
- Highways
- Canal
- Snake River
- County Boundary
- Municipality
- Lakes/Reservoirs
- Contours 100ft
- Eastern Snake Plain Aquifer

A & B Irrigation District
Potential Gravity Diversion Pipeline Alignment

Figure 3

0 3.5 7 14 21 28 Miles

Date: June-2008

Projection: Idaho Transverse Mercator - Meters
Datum: North American 1983
Source: District Boundaries and irrigated area, IDWR September 2005.

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Conceptual Alternative 2 – Pressurized Diversion

The second conceptual alternative involves developing a pressurized distribution system for Unit B. There are numerous options to consider in this alternative, including the location and number of potential intakes, the configuration of the pipelines, and the use of open channels and storage. Potential options were identified and subsequently evaluated at a conceptual level to determine the best option of Conceptual Alternative 2.

Initial Conceptual Alternative Development and Evaluation

Preliminary evaluation of this alternative resulted in the development of five potential options for initial consideration. The options are as follows:

OPTION 2a: Surface water would be pumped from Lake Walcott into a single main pipeline for conveyance into Unit B. Laterals from this main pipeline would distribute the water to the groundwater well locations;

OPTION 2b: Surface water would be pumped from Lake Walcott into two main pipelines for conveyance into Unit B. One main pipeline would serve the northern half of Unit B, while the other would serve the southern half of Unit B. Laterals from the main pipelines would distribute the water to the groundwater well locations;

OPTION 2c: A dual distribution system would be used to pump surface water from Lake Walcott into the eastern portion of Unit B and from the Snake River west of Burley into the western portion of Unit B. Laterals from the main pipelines of each system would distribute the water to the groundwater well locations;

OPTION 2d: Surface water would be pumped from Lake Walcott into a canal running roughly east to west through Unit B. Water would be pumped or gravity diverted from the canal into laterals which would distribute the water to the groundwater well locations;

OPTION 2e: This option would incorporate a storage component into OPTION 2a, 2b, 2c or 2d to reduce the required distribution system capacity; and

OPTION 2f: Surface water would be pumped from Lake Walcott into the northern main pipeline envisioned in OPTION 2b. Laterals from the main pipeline would distribute the water to the groundwater well locations in the northern half of Unit B. Lands in the southern half of Unit B would continue to be served by existing groundwater wells.

After considerable discussion with the Technical Advisory Committee, it was decided to focus further evaluation on OPTION 2a, 2b and 2c; with the best option being subjected to a surface storage sensitivity analysis as envisioned in OPTION 2e to determine the effect of storage capacity on distribution system capacity.

OPTION 2d was eliminated from further consideration for multiple reasons. First, it was determined that the canal would have to be lined to reduce seepage because it was assumed that

any available supply of surface water would be limited. Lining the canal would eliminate the cost benefit of an unlined canal compared to the pipeline options. Second, evaporation losses would continue to be a problem even after lining the canal. Third, after reviewing the undulating topography within A&B (Figure 4), it was uncertain that a gravity canal could be implemented effectively. At a minimum, it would require large cut and fill operations or an extremely sinuous alignment that would likely exacerbate the evaporation losses. Lastly, a canal would require a relatively large footprint which could take irrigated land out of production.

OPTION 2f was eliminated from further consideration at this time because it did not fully meet the scope of the study, which was to fully convert Unit B to surface water. In addition, this option would rely on the incidental recharge from the surface water deliveries in the northern portion of Unit B to provide future supply for the southern half of Unit B. The analysis of Conceptual Alternative 3, Groundwater Recharge, will need to occur prior to determining the merit of this option.

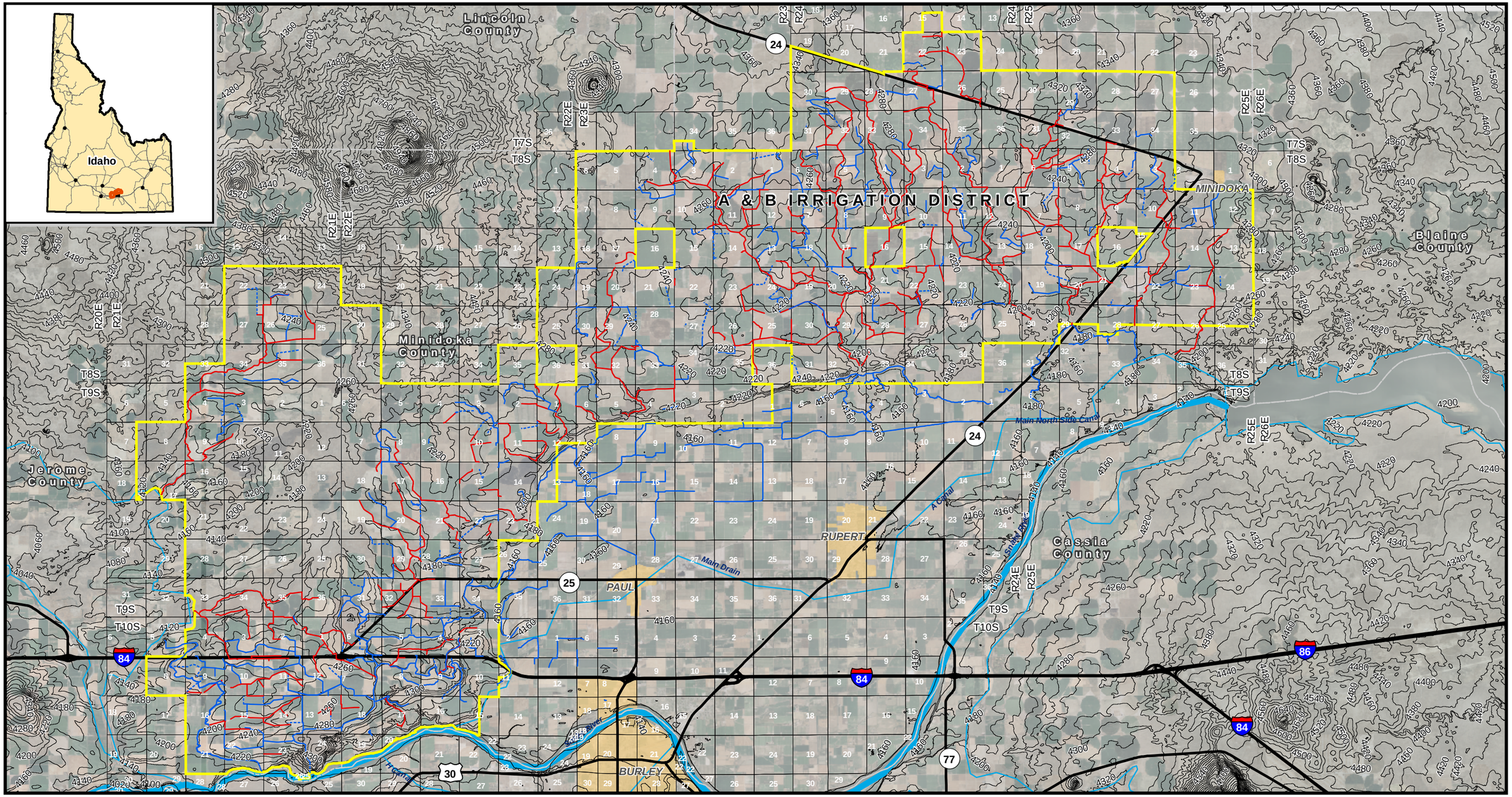
Although these options were discarded at this point for the conceptual purposes of this study, further analysis may be warranted in the future.

Refined Conceptual Alternative Evaluation

As stated above, OPTION 2a, 2b and 2c were identified for additional analysis to assist in determining one pressurized diversion option for final conceptual analysis. Refer to Figure 5 for a conceptual layout of the three options. To initiate this effort, a preliminary cost comparison of the main pipelines from each remaining option was conducted that only included pipeline and pump station costs.

In order to determine main pipeline and pump station capacities for this preliminary estimated cost comparison, a conceptual design process with simplifying assumptions was used. For example, because the topography of Unit B is relatively level with slight undulations in the east-west direction, the main pipelines were assumed to be level. A calculation of the difference in elevation between the groundwater well locations and the minimum Lake Walcott elevation (4236 feet) averaged 15 feet and was considered minimal in comparison to frictional losses. Therefore head requirements were based only on frictional losses for OPTION 2a and 2b, the single diversion point alternatives. For OPTION 2c, with a second intake along the Snake River west of Burley, the difference in elevation between the Snake River and the corresponding groundwater well locations in the western region of Unit B averaged 85 feet. To account for a greater elevation lift required by the pumps at the second intake, 70 feet of head (85 feet – 15 feet) was included to make the analysis more comparable between the three options.

The resulting costs of the main pipelines and pump stations for all three options were very similar, with a difference of only ten percent. Because of this, the preliminary analysis did not indicate a clear choice, with the cost differences easily falling within the margin of error for such a rough comparison. Therefore, a more qualitative analysis of the three alternatives was performed as another means of comparison.



Legend

- Drain
- Lateral
- Pipeline
- A & B ID Boundary
- Section Boundary
- County Boundary
- Municipality
- Canal
- Snake River
- Highways
- Contours 20ft

A & B Irrigation District
Current Conveyance System and Topography

0 1 2 4 6 8 Miles

Date: June-2008

Projection: Idaho Transverse Mercator - Meters
Datum: North American 1983
Source: District Boundaries and irrigated area, IDWR September 2005.

Figure 4

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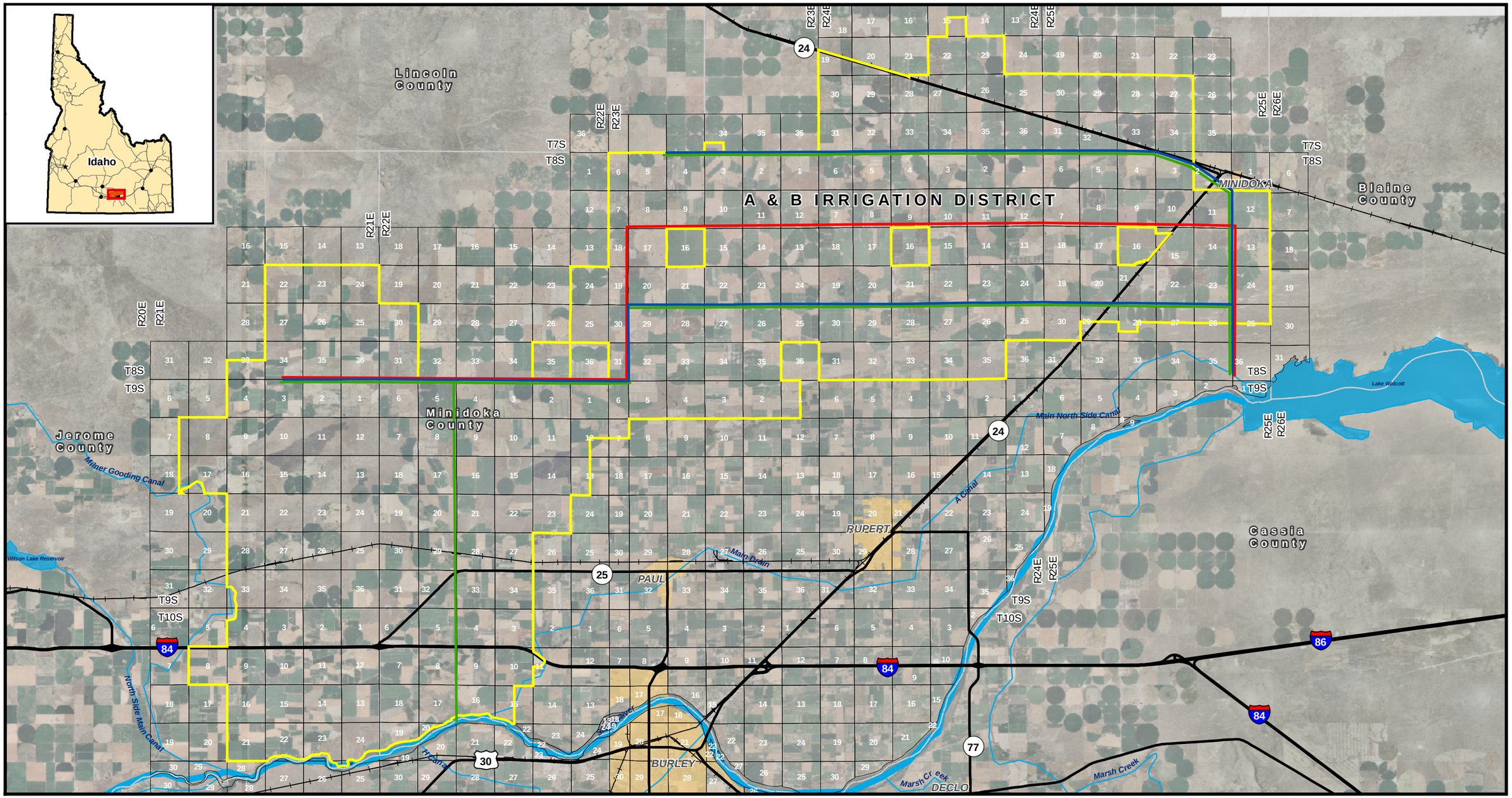

Legend

Alternative 2 Alignments

- Option 2a
- Option 2b
- Option 2c

- Highways
- Railroad
- Contours 20ft
- Canal
- Snake River

- A & B ID Boundary
- County Boundary
- Municipality
- Section Boundary
- Water Body

A & B Irrigation District
Alternative 2 Pressure Distribution System
Alignment Options

0 0.5 1 2 3 4 Miles

Date: June-2008

Figure 5

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Projection: Idaho Transverse Mercator - Meters
Datum: North American 1983
Source: District Boundaries and irrigated area, IDWR September 2005; Image, USDA NAIP 1-meter Digital Orthophotography 2006.

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Initial pipe sizing of the main pipelines showed pipes ranging from 14 feet for OPTION 2a, 10 feet for OPTION 2b, to 9.5 feet for OPTION 2c. With transportation issues becoming a serious concern with diameters approaching the 12-foot range, OPTION 2a immediately became questionable. The construction of an on-site shop and the necessary equipment, manpower and resources for a limited amount of pipeline fabrication does not appear reasonable; and consequently, OPTION 2a was eliminated from further consideration.

Other criteria in the qualitative comparison of the options were power transmission and right-of-way crossings. Based on the capacities of the pump stations for the 1000 cfs system, power requirements would require bringing in additional power to the main pump station site(s). For this evaluation, it was assumed power for the Lake Walcott pump station would be brought directly from Minidoka Dam. For OPTION 2c, with two intake sites, additional power would have to be brought to the pump station located along the Snake River west of Burley. Although power availability is subject to major fluctuations based on existing demands and infrastructure, currently power would have to be brought from roughly 6 to 7 miles away with substation upgrades. With a second intake west of Burley, OPTION 2c also would require boring under Interstate 84 to reach the northwest portion of Unit B. Both the additional power transmission and interstate crossing would add additional costs to OPTION 2c that are not required for OPTION 2b. Although OPTION 2c is still a viable option that may be considered in further detail in the future, at this level of conceptual study it appears OPTION 2b, with one point of diversion and two main pipelines, is the best option of Conceptual Alternative 2.

Conclusions

With the identification of OPTION 2b as the best option of Conceptual Alternative 2, further analysis was performed to develop the option into a more complete alternative.

During a site visit, the Minidoka North Side Canal was identified as having the potential to divert water from Lake Walcott to a point closer to the southern edge of A&B (Refer to Site Visit Report – Appendix B). The Minidoka North Side Canal is located on the north end of Minidoka Dam and currently has a maximum capacity of 1,700 cfs (Reclamation 1961). Permission from the Minidoka Irrigation District (owner of the Minidoka North Side Canal) and Reclamation will be required before the analysis is taken further than the conceptual level presented in this study. It should be noted that Reclamation is currently considering replacement of the headworks, although no increase in capacity is being considered at this time (Reclamation 2008a).

Assuming the operating level of Lake Walcott is maintained at or near the maximum elevation of 4245 feet during the irrigation season, the existing headworks of the Minidoka North Side Canal should have adequate capacity to accommodate the existing maximum flow plus the additional 1,000 cfs for the A&B pressurized distribution system. However, in order to utilize the Minidoka North Side Canal, a pumping plant and intake must be placed approximately 0.75 miles downstream of the canal headworks and the canal would need to be enlarged from the headworks to the proposed pumping plant and intake to allow for the additional 1,000 cfs flow. In order to maintain existing elevation head differences between Lake Walcott and the Minidoka North Side Canal, it was opted to expand the width rather than the height of the canal for this evaluation. This would involve widening the canal from the existing 30-foot width to

approximately 55 feet and would require demolition and reconstruction of two bridges that cross over it (Reclamation 1907).

From the proposed pumping plant on the Minidoka North Side Canal, two 10-foot pipelines would deliver water into Unit B. One main pipeline would deliver approximately 500 cfs to the northern half of Unit B, while the other would deliver approximately 500 cfs to the southern half of Unit B. Laterals from each main pipeline would deliver water to the existing groundwater delivery systems for subsequent delivery to growers.

This pressurized distribution system would also include 1,000 acre-feet of storage (12 hours of supply at the maximum delivery rate) located at unspecified points within the system to improve operational flexibility and lessen the effects of system peaking.

Please refer to Figures 6 and 7 for the conceptual layout of the distribution system within A&B and the conceptual pumping plant/intake on the Minidoka North Side Canal.

Conceptual Alternative 3 – Groundwater Recharge

The objective of the aquifer recharge alternative would be to enable the continued use of existing irrigation production wells throughout A&B, without constructing an extensive surface water distribution system as would be required for Conceptual Alternative 2.

Recharge Method Descriptions and Considerations

Three primary methods of recharge have been considered for this study. These options include the following:

OPTION 3a: Recharge by means of shallow infiltration basins;

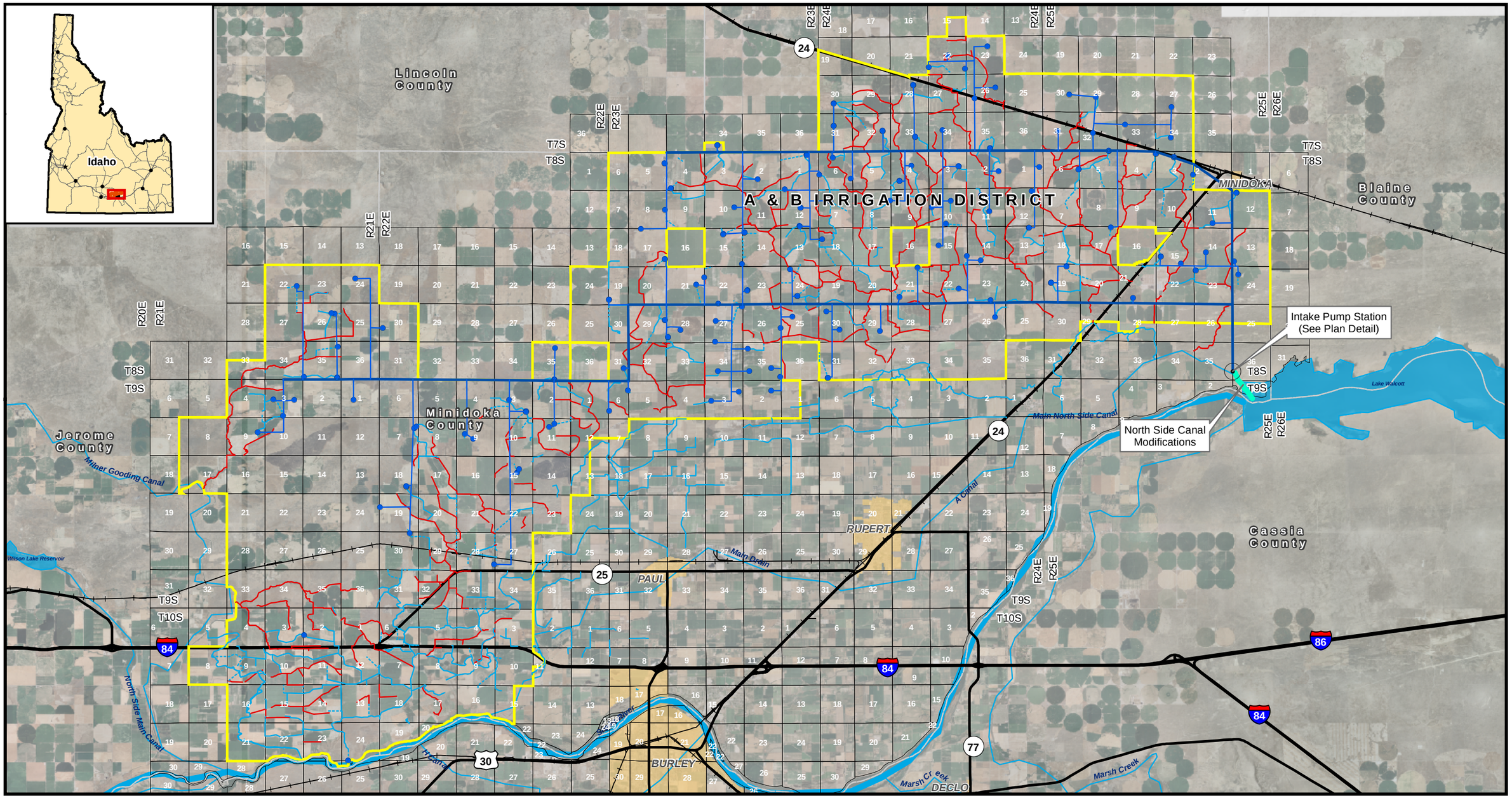
OPTION 3b: Injection into vadose zone wells; and

OPTION 3c: Aquifer injection wells.

For the purposes of this study, it is assumed that the intake, pump station, and main pipeline system to convey water from Lake Walcott to the points of recharge would be essentially the same as in OPTION 2b. In the case of infiltration basins, it is likely that less pipeline would actually be required than for OPTION 2b; whereas vadose zone injection wells or aquifer injection wells may not substantially reduce the amount of pipeline because of a wider required distribution. Each of these methods is discussed below. Additional discussion of recharge methods and considerations is provided in Appendix C.

Evaluation of Recharge Potential

The viability of aquifer recharge in A&B depends on the surface and subsurface conditions within the Unit B study area. An overriding consideration for any recharge program is the anticipated level of effort, and associated costs, required to implement the program. Therefore if



Legend

Distribution Network
- Main
- Lateral
- Active Well

Existing Drains, Laterals & Pipelines
- Drain
- Lateral
- Pipeline

- Highways
- Railroad
- Contours 20ft
- Canal
- Snake River

- A & B ID Boundary
- County Boundary
- Municipality
- Section Boundary
- Water Body

A & B Irrigation District
Proposed Pressure Distribution Network

0 1 2 4 6 8 Miles

Date: June-2008

Figure 6

N

Projection: Idaho Transverse Mercator - Meters
Datum: North American 1983
Source: District Boundaries and irrigated area, IDWR September 2005; Image, USDA NAIP 1-meter Digital Orthophotography 2006.

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File: letter fig - pump station.dgn Model: Default ColorTable: bw.ctb DesignScript: MWH_Iplot_PenTable_V85.pen PlotScale: 20:1 Plot Date: 02-JUN-2008 10:05
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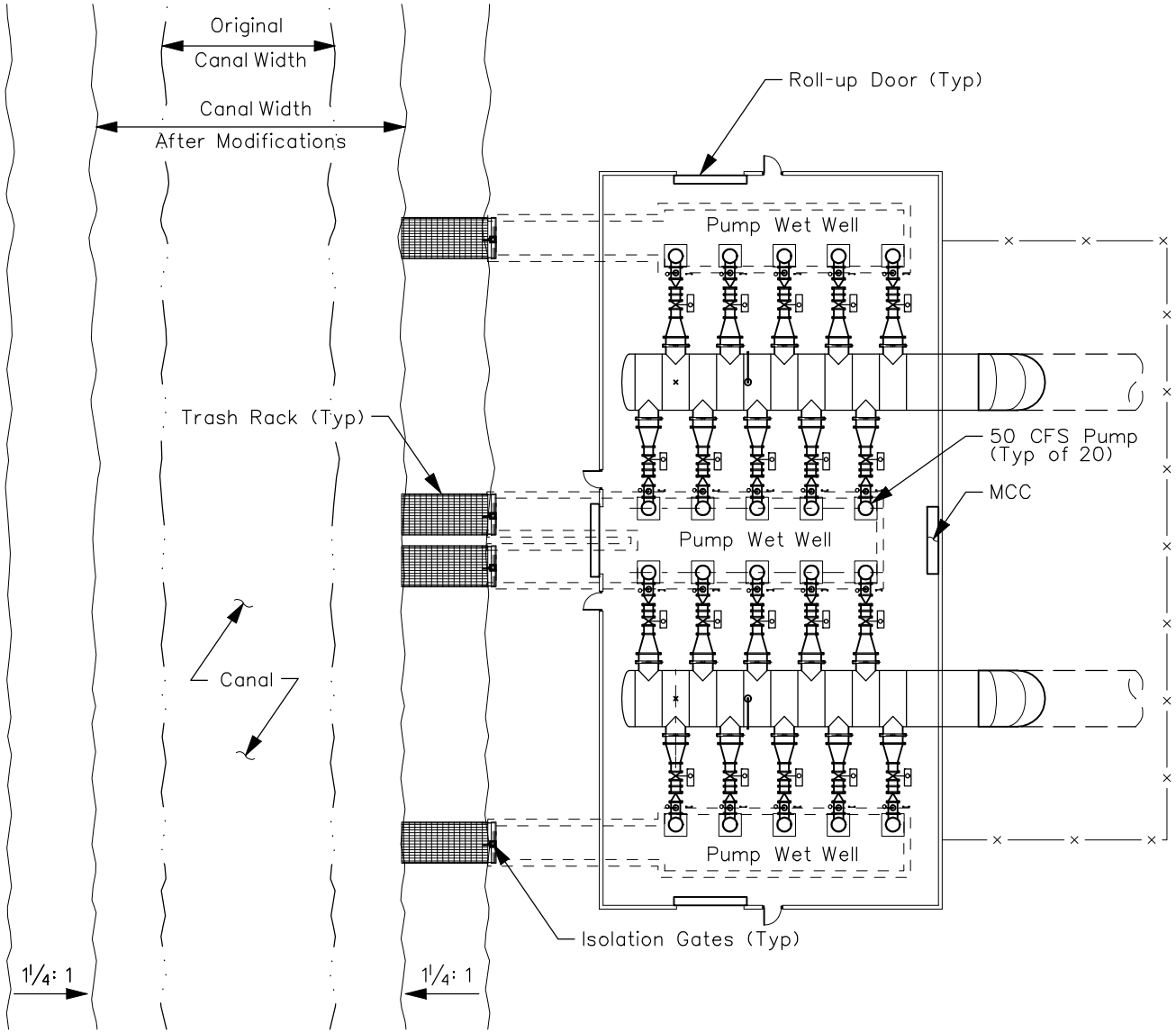

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1000 CFS Conceptual Intake/Pump Station
with Canal Modifications
Figure 7

a recharge program is implemented, it must be cost-effective relative to the other available options. Although no effort is made here to provide a detailed opinion of probable costs for aquifer recharge, a discussion of relative costs is included for comparative purposes.

Evaluation of the technical feasibility of aquifer recharge was performed by reviewing data from the following sources:

- Well driller reports (well logs)
- Previous studies, including Reclamation (2003) and Ralston (2008)
- Soils maps and reports (NRCS 1975)
- Soil grab samples
- Snake River flow records (USGS minimum mean daily flows and mean monthly flows, 1977 to 2007).

Please refer to Appendices C and D for additional discussion of the sources listed above.

Feasibility of Recharge

OPTION 3a - Infiltration Basin Recharge: Because infiltration basin recharge is usually the most economical way to achieve aquifer recharge, it is often the preferred alternative. However, because of fine-grained surface soils throughout the study area, its implementation may be impractical in this instance. A preliminary estimate of infiltration capacity of the surface soils suggests that from 2,600 to 3,900 acres of land would be needed to recharge enough for complete replacement of current irrigation water demand, based on the historical wellfield capacity of 225,000 acre-feet per year. Furthermore, where fine-grained subsurface layers and/or thick layers of unfractured basalt are present, vertical infiltration would be impeded and further interfere with recharge. Subsurface layers of low-flow material are present throughout much of the site. The data indicate that the northeast part of the study area would be most favorable for recharge because the surface soils are generally not as thick as elsewhere, and subsurface obstacles to vertical infiltration appear to be less extensive. However, it is likely that even in this area, thick sequences of unfractured basalt flows would limit the recharge capacity within the confines of the study area. A more complete discussion of the evaluation of infiltration basin recharge is provided in Appendix C.

OPTION 3b - Vadose Zone Injection Wells: Vadose zone injection wells may or may not be technically feasible depending on the presence of subsurface vertical flow barriers throughout much of the area. If implemented, injection wells would need to be deep enough to inject recharge water below shallow silt/clay layers and massive basalt flows. Well logs suggest that, on average, vadose zone wells would need to be 50 to 100 feet deep, or more, to achieve this objective. For the purposes of this evaluation, it is assumed that vadose zone injection wells would be 100 feet deep.

Well logs show that production rates from existing wells are high in most instances. Flow rates of two to ten cfs (900 to 4500 gpm) are reported on many well logs. Typically, injection wells are not able to recharge water at the same rate as withdrawal pumping. MWH experience has

been that injection wells can usually be operated successfully at about one-half the production capacity. For this evaluation, an average injection rate of 1,000 gpm was assumed.

For the projected six-month recharge period to recharge up to the full water rights of approximately 225,000 acre-feet (historical capacity) at an average rate of 625 cfs (281,000 gpm), approximately 280 vadose zone injection wells would be required. At a cost of $200,000 per well (based on MWH experience), the cost of well construction would be more than $50 million. The wells would probably be distributed throughout much of Unit B to prevent excessive perching or mounding of groundwater, so it is unlikely that much savings would be achieved through reduced distribution pipeline costs. A longer recharge period would reduce the total number of required wells, but the number would still be very large and the distribution system would be more extensive than for recharge basins.

OPTION 3c - Aquifer Recharge Injection Wells: Strictly considered from a technical perspective, aquifer recharge by means of injection wells has a high probability of success. The high production capacities of many of the existing wells suggest that recharge could be achieved if injected directly into the aquifer. As with vadose zone injection wells, however, substantial maintenance issues may be expected, and the wells would need to be widely distributed to be most effective. This would offset most of the savings when compared to surface water replacement, because an extensive distribution pipeline system would be required. Treatment of raw water prior to injection would almost certainly be required for direct injection under the State of Idaho's Underground Injection Control program to protect groundwater quality. Because blending of surface water and groundwater of dissimilar chemistries can result in various problems such as precipitation of minerals, changes in pH, dissolution of minerals in the formation, and other issues, additional treatment (at additional cost) may be necessary to mitigate these effects.

A typical aquifer recharge injection well would need to target the same aquifer layers currently used for production. Therefore a typical injection well would need to be about 300 feet deep or more in most instances. From MWH experience, these wells would cost $600,000 to $800,000 to construct. The number of injection wells would be the same as has been estimated for vadose zone injection wells (approximately 280 for a six-month recharge period). At $600,000 per well for construction, well construction would likely cost more than $160 million. When considered together with the cost of raw water treatment and required pipeline distribution facilities, the cost of direct aquifer recharge injection wells becomes prohibitive, or at least more expensive than surface water replacement.

Conclusions

Aquifer recharge by means of surface infiltration recharge basins (OPTION 3a) is estimated to require approximately 2,600 to 3,900 acres of land because of the low permeability of surface soils. Surface conditions suggest that recharge basins would have the highest probability of success if concentrated in the northeastern part of Unit B, near Highway 24 northwest of the town of Minidoka. However, it is not clear whether subsurface conditions would be very favorable for recharge from basins anywhere within A&B. Further evaluation may be warranted,

but it is questionable whether surface infiltration recharge basins are the most practical approach for this area.

Recharge by means of vadose zone injection wells (OPTION 3b) has a higher probability of success than recharge basins. However, vadose zone injection well recharge would require much higher costs and would be achieved by a large network of injection wells that would likely offset any savings when compared to surface water replacement. Therefore, vadose zone injection well recharge does not appear to be a cost-effective approach to recharge in this instance.

Aquifer recharge injection wells (OPTION 3c) have a higher probability of successful recharge than either recharge basins or vadose zone injection wells. However, the very high costs associated with injection well construction, surface water distribution, and required treatment of raw surface water before injection, would prohibit this option from implementation for an irrigation water system.

Aquifer Storage and Recovery (ASR), using the existing production wellfield to inject and recover water from the same wells, was not evaluated in any detail but also appears to be impractical as a means of recharge to groundwater for irrigation water systems because of the high cost and the potential impacts to existing production wells.

RECOMMENDED CONCEPTUAL ALTERNATIVE – PROJECT COST

Based on the preliminary evaluation of Conceptual Alternatives 1, 2 and 3, OPTION 2b of Conceptual Alternative 2 appears to have the most merit and was selected as the recommended conceptual alternative. As described above, this option consists of a pressurized distribution system from a single intake source off of the Minidoka North Side Canal originating at Lake Walcott. Conveyance into Unit B is accomplished through two main pipelines; one serving the northern half of Unit B and the other serving the southern half of Unit B. Refer to Figures 6 and 7 for a conceptual layout of the recommended distribution system and intake.

PROJECT COSTS

Additional analysis of OPTION 2b was performed to develop an International Class 5 opinion of probable project and operational costs as defined by the Association for the Advancement of Cost Engineering (AACE). From the conceptual layout shown in Figures 6 and 7, pipeline sizes and lengths and pumping station capacities were determined and costs were quantified. Included in the pipeline capital cost opinions were associated valves, earthwork, and borings; included in the pump station capital cost opinions were associated SCADA and power transmission. In addition, cost opinions for the Minidoka North Side Canal expansion and the regulating storage were included in the overall opinion of probable project cost. The required labor and equipment to install the above items were also included.

A summary of the Class 5 opinion of probable project cost is presented in Table 1. Refer to Appendix E for the actual Class 5 opinion of probable project cost, the definition of a Class 5 opinion of probable project cost and the design assumptions and criteria that went into

developing the opinion. All cost opinions are conceptual-level costs presented in 2008 dollars. The maximum accuracy range is from -50% on the low side to +100% on the high side. The final costs will depend on actual labor and material costs, competitive market conditions, implementation schedule, and other variable factors. As a result, the final project costs will vary from the opinions presented herein.

TABLE 1

PRESSURIZED DISTRIBUTION OPINION OF PROBABLE PROJECT COST

Cost Item	Cost Opinion (2008 Dollars)
Minidoka North Side Canal Modifications	$1,100,000
Pipeline (Includes valves, earthwork, & borings)	$268,500,000
Pump Stations (Includes power & SCADA)	$33,500,000
Regulating Storage	$2,400,000
SUBTOTAL	$305,500,000
Estimating Contingency @ 15%	$45,800,000
ROUNDED BASE CONSTRUCTION COST	**$350,000,000**
Engineering, Administration, Environmental, Construction Services	$10,000,000
OPINION OF PROBABLE PROJECT COST	**$360,000,000**

It is the opinion of MWH that the project cost identified above represents the most appropriate cost figure at this conceptual level of study given the accuracy range of a Class 5 opinion of probable project cost. As discussed in greater detail in Appendix E, numerous unknown factors can influence the accuracy of this opinion. Among them are price increases, market volatility and undefined or unknown regulatory or environmental requirements. To evaluate the potential for these and other uncertainties to negatively affect the cost of this project, a scope contingency of 20 percent could be added to the opinion of probable project cost. This would add approximately $70 million to the total project cost opinion. On the other hand, the bidding environment can be extremely competitive when dealing with large material orders as envisioned in this project, resulting in lower than expected contractor bids. Again, it appears the opinion of probable project cost identified in Table 1 reflects the most appropriate Class 5 cost opinion figure at this point in time.

OPERATION & MAINTENANCE (O&M) COSTS

Operation and maintenance costs for the pressurized distribution system consist of power, labor, and system maintenance costs. Power costs were based on a rate of 19.96 mills reported as the 2008 inflated price from the Reclamation based on the 2007 rate of 19.3 mills (see Appendix F). The preliminary annual energy usage projection to pump 178,000 acre-feet (the 1995-2007 average annual groundwater production volume) through the pressurized distribution system was 44,800,000 kilowatt hours (kWh). Staffing was based on experience with irrigation systems of

similar layout, size, and SCADA capabilities. Staffing costs were based on nine positions consisting of a general manager, financial supervisor, operations supervisor and six maintenance workers. Annual system maintenance costs were assumed to be roughly 0.5 percent of the total base construction capital costs. Annual O&M costs are summarized in Table 2.

TABLE 2

PRESSURIZED DISTRIBUTION O&M COST

O&M Item	**Cost Opinion (2008 Dollars)**
Power	$900,000
Labor	$700,000
Maintenance	$1,800,000
TOTAL ANNUAL O&M COST	**$3,400,000**

It is expected that labor requirements for the pressurized distribution system would be comparable to the existing A&B groundwater system, with potential savings given the proposed SCADA system. However, this could not be verified due to limitations in obtaining existing A&B data.

For power costs, total annual energy usage in 2007 by existing Unit B groundwater wells was reported to be 89,700,000 kWh with a pumped volume of 184,000 acre-feet (Refer to Appendix F). Comparing the 2007 power figures from the existing groundwater system with the projected pressurized surface water distribution system power figures indicates that the pressurized system uses roughly half the energy of the existing system, resulting in a savings of half of the existing annual power costs.

COST SENSITIVITY ANALYSIS AND DISCUSSION

Storage

The major cost in the pressurized distribution system is the main pipelines, primarily because of their large size. To reduce the size of the main pipelines, storage could be incorporated into the design at the lower end of the system. During periods of high demand, the lower end of the distribution system would be supplied from storage, thus allowing the upstream main pipelines to be reduced in size. The storage reservoirs would be filled to capacity during the early irrigation season when demands are low and excess capacity exists in the distribution system.

A storage reservoir capable of supplying the northern and southern main pipelines was assumed to be generally located near the end of the northern pipeline. Volumes of 5,000, 10,000, and 15,000 acre-ft were preliminarily evaluated to determine potential impacts on capital costs. Reductions in base construction costs with storage fell roughly at 4, 7, and 9 percent for storage volumes of 5,000, 10,000, and 15,000 acre-ft, respectively. Greater savings may be possible if

two smaller reservoirs are used, each serving one main pipeline, rather than one large reservoir serving both main pipelines. This would allow the reservoir serving the northern pipeline to be located further upstream so that the main pipeline at the bottom of the northern system does not need to be increased in size to accommodate reverse flow from the reservoir. More detailed analysis on storage size and location is warranted if this alternative is pursued further.

ANNUALIZED COST ANALYSIS

In order to determine an annual overall project cost, including capital and O&M costs, a financing rate of 6 percent and term of 30 years were assumed. Using these financing assumptions, the capital cost of the project was amortized and added to the O&M costs outlined above. The annual project cost is summarized in Table 4.

TABLE 3

ANNUAL PROJECT COST

Project Cost Component	**Cost Opinion (2008 Dollars)**
Annual Capital Cost	$26,200,000
Annual O&M Cost	$3,400,000
TOTAL ANNUAL PROJECT COST	**$29,600,000**

[1]Annual capital cost includes base construction, engineering, administration, environmental and construction services costs.

RECOMMENDATIONS

Conceptual Alternative 2, OPTION 2b is the recommended alternative at this level of study. While the project costs appear high for an irrigation delivery project, MWH recommends that additional study of this alternative be conducted once an available water supply is identified and quantified. As the study indicates, the main pipelines of the distribution system constitute the largest portion of the project cost and any reduction in system capacity will have a resulting reduction in the overall cost of the system. Therefore, it is imperative that the system is sized to a capacity that optimizes the timing and magnitude of the actual water supply to be delivered. In addition, strategically located storage facilities could offset the required size of the main pipelines as well, and should be more comprehensively evaluated in a future study.

Although this evaluation was based on an assumption that 100 percent of the current design flow would be available from surface water diverted from the Snake River, it is unlikely that this amount of water would be available in most years. The anticipated supply for surface water replacement would come from a combination of new surface water storage associated with a propose raising of Minidoka Dam, together with an exchange of water rights for salmon flows. It is estimated that these sources would be able to provide full replacement of A&B water in about 50 percent (or less) of the yearly irrigation seasons. The remaining years would get

something less than full replacement, and some years may result in little or no surface water available for A&B irrigation use.

If the amount of available water supply is inadequate to meet the total irrigation demand of Unit B, a conjunctive use operation could be implemented where surface water is supplied when available and groundwater is supplied when surface water is unavailable. A&B may be able to realize operational efficiencies with this type of a program and the availability of a surface water distribution system may allow access to surplus water in the non-irrigation season for storage in reservoirs or in the aquifer if further field exploration determines that groundwater recharge is a viable option.

Many of the options that were preliminarily ruled out of consideration in this conceptual study based on simplified criteria and assumptions deserve additional evaluation. MWH recommends consideration of the feasibility of providing partial surface water replacement to Unit B as described in Conceptual Alternative 2, OPTION 2f. This proposed alternative will eliminate as much as half of the groundwater withdrawals by providing surface water to the northern half of Unit B and has the potential to provide incidental recharge from the surface water irrigation deliveries upgradient of a majority of the remaining groundwater wells. In addition, less surface water supply would be required and the capital cost of the surface water distribution system could be substantially less than the recommended conceptual alternative opinion of probable project cost presented in this study. Determining the incidental recharge benefit of the irrigation deliveries will be necessary to evaluate the net effect of this alternative on the ESPA. If this option is considered, it is recommended that exploratory drilling and detailed soil analysis be conducted in the northeast part of Unit B.

MWH recommends that an optimization analysis be performed to determine the most cost-effective system size and approach. This would apply to both the conjunctive use approach as well as the partial replacement approach. An optimized pipeline and pump station for either approach would result in substantial reductions in capital costs when compared to the opinion of probable cost prepared for this evaluation. This optimization should consider the O&M costs associated with operating, maintaining, and periodically rehabilitating the existing wellfield as well as the proposed surface water replacement options.

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REFERENCES

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Appendix A

Appendix A: Ground Water Production Records Summary

	April-May	May-June	June-July	July-Aug	Aug-Sept	Sept-Oct	Annual Usage
1995	5331	11817	43589	44418	34859	20206	160220
1996	19586	24239	52904	41925	33985	17551	190190
1997	14651	38026	48221	32921	27993	12459	174271
1998	12221	14501	45177	41710	34523	12122	160254
1999	3640	28095	52141	41365	30350	19427	175018
2000	22483	41521	53126	41272	32145	16541	207088
2001	16859	41039	52005	40698	32249	13518	196368
2002	11718	33966	52075	38324	31859	14692	182635
2003	1876	38948	53315	41491	30861	16570	183059
2004	23724	35526	48892	35384	25346	13049	181921
2005	1552	15740	46312	39608	33856	13095	150163
2006	7625	30409	49855	39567	35167	9903	172525
2007	17082	38278	51245	36188	32041	9001	183834
AVG	**12181**	**30162**	**49912**	**39605**	**31941**	**14472**	**178273**
MAX	**23724**	**41521**	**53315**	**44418**	**35167**	**20206**	**207088**

** Volumes given in acre-ft*

Appendix B

A&B Irrigation District

Groundwater-to-Surface Water Conversion Project
Site Visit Notes: John Roldan, MWH
April 24, 2008

1. The first stop of the site visit was at Minidoka Dam to view the Minidoka North Side Canal (NSC) intake and the area around the intake for potential new intake sites. At the facility, I met with the following USBR Minidoka Dam personnel: 1) Brett Barnhart, Facility Manager, and 2) Kevin Shomaker, Superintendent. Kevin showed me around the NSC intake and informed me that he believes there is unused capacity available. Even in the middle of the irrigation season, he has not seen Minidoka Irrigation District (MID) open all of the nine slide gates. Kevin also indicated that MID has discussed moving the intake to a bridge location approximately 50 yards downstream. Whether the existing intake is used or a new intake is constructed downstream, the channel immediately downstream of the existing intake will need to be enlarged to prevent flooding of the USBR Operations Office. While all of this must ultimately be verified, discussed and approved by MID and the USBR, utilizing the NSC intake and enlarging a small portion of the downstream canal seems to make sense at a conceptual level. Kevin provided me with the proper contacts at the USBR Snake River – East office to locate drawings of the NSC intake.

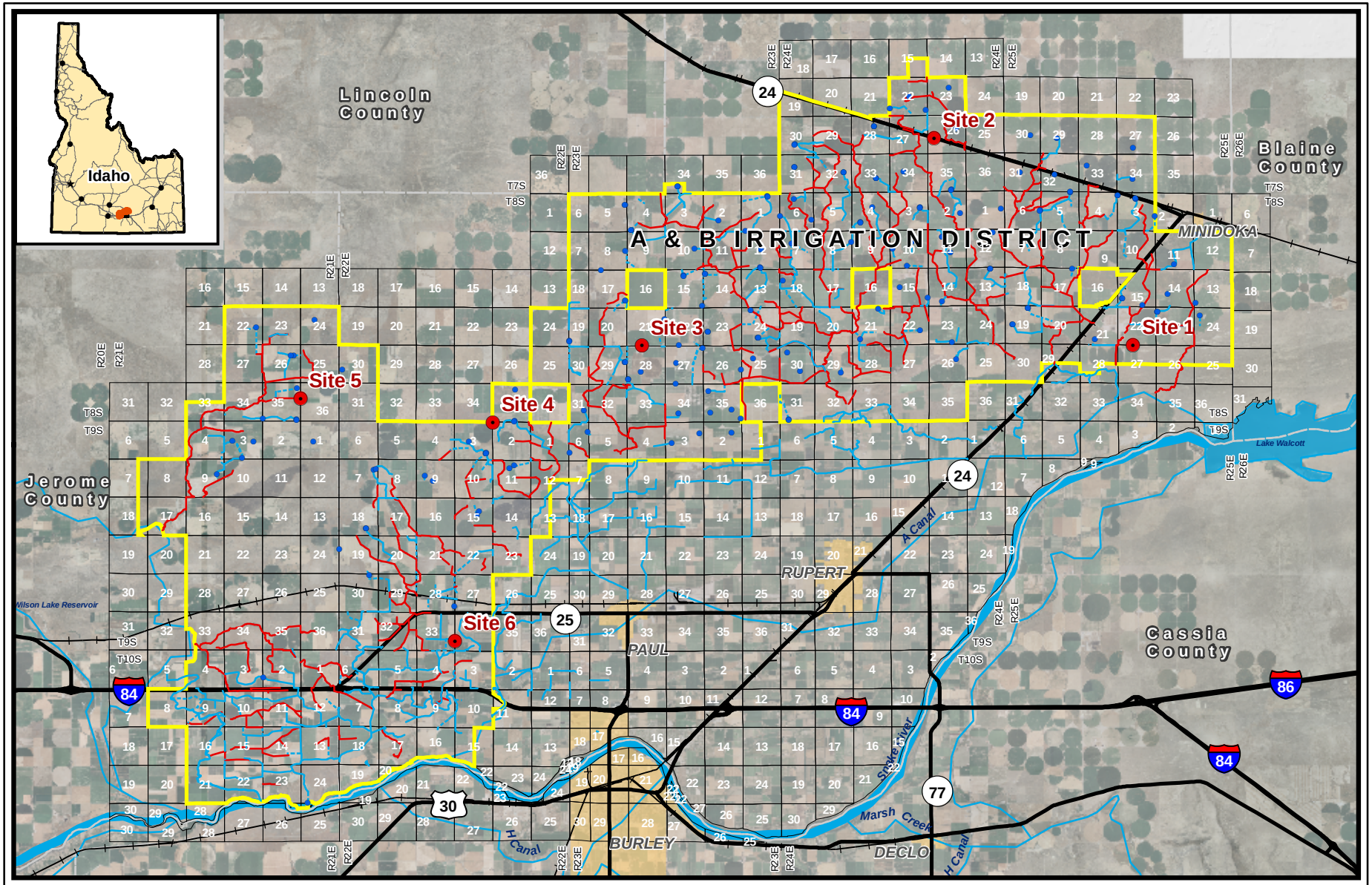
   A potential site for a new intake is directly north of the NSC intake on the other side of the existing electrical switchyard. Dredging may be required to ensure the intake has adequate capacity. The new intake and associated canal or pipeline alignment would be on the south end of Lake Walcott State Park.

2. The site visit then continued with a drive through the A&B Irrigation District (A&B) to familiarize myself with topographic and manmade obstacles. Highway 24 and the Union Pacific Railroad that parallels its path appear to be unavoidable obstacles that may require special construction measures to cross. The topography of the district is characterized by slight undulations in the eastern portion of the district with a few areas of slightly sharper relief in the western portion of the district north of Unit A. No particular site appeared to have the topographic qualities to be easily converted into a reservoir, although the area north of Unit A may warrant further investigation.

   Existing groundwater well systems within the A&B appear to be of two types: 1) wells that discharge into catch basins which serve gravity laterals controlled by weir boards and slide gates and 2) wells that are connected by manifold into discharge pipelines.

   Small topsoil samples at six locations throughout the A&B were also obtained to verify the soil types indicated in the Natural Resource Conservation Service soil maps (Refer to Figure 1 for locations).

3. Finally, diversion locations along the Snake River were investigated. The area around the existing Unit A pumping plant and the area approximately one-half mile



Legend

- Soil Sample Locations
- Active Well
- Drain
- Lateral
- Pipeline
- A & B ID Boundary
- Section Boundary
- Highways
- Railroad
- County Boundary
- Canal
- Snake River
- Municipality
- Water Body

0 1 2 4 6 8 Miles

A & B Irrigation District Soil Sample Locations

Figure 1

Projection: Idaho Transverse Mercator - Meters
Datum: North American 1983
Source: District Boundaries and irrigated area, IDWR September 2005; Image, USDA NAIP 1-meter Digital Orthophotography 2006.

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west of the eastern border of A&B (approximately 3 miles from the western edge of the town of Burley) appear to be adequate diversion sites. Both are within the Milner Dam reservoir pool. The problems with the sites include the need to pump over adjacent hills, the need to cross Highway 84 to get to Unit B, and the need to bring in power, or increase the size of the existing transmission lines, to the remote sites.

Minidoka North Side Canal intake structure at Minidoka Dam

Looking downstream from Minidoka North Side Canal intake structure

Potential site for new intake structure

Looking upstream at Minidoka North Side Canal from bridge at 400 North Road

Existing groundwater wells and catch basin

Catch basin with gravity laterals

Groundwater well system with manifold and discharge pipelines

Potential diversion site approximately 3 miles west of Burley (opposite side of Snake River)

Appendix C

APPENDIX C

RECHARGE METHOD DESCRIPTIONS AND CONSIDERATIONS

General

If feasible, aquifer recharge might have some advantages over other alternatives for the A&B program. The required pipeline network could be less extensive than would be necessary for complete replacement using surface water because water would only have to be conveyed to the points of recharge, rather than to each existing individual groundwater delivery system. Little modification to the existing system of wells and ditches would be necessary because existing wells, pumps, and conveyance ditches would remain in use. Because recharge could occur over a longer period than surface replacement (before, during, and after the irrigation season), the diversion rate could be smaller and therefore the pipelines could be smaller as well, which would reduce costs. The pumps at the pump station also could be smaller, reducing capital costs for pumps.

Aquifer recharge requires certain favorable geologic conditions to be successful. Those requirements depend upon the method of recharge used. The objectives of aquifer recharge, the advantages and disadvantages of different methods, the conditions required for successful recharge, and the suitability of the study area for each method, are discussed hereafter.

All recharge projects that divert surface water for groundwater storage are dependent on the availability of surface water and may be subject to restrictions on the rate and time of diversion. For example, if minimum streamflows must be maintained to ensure availability for downstream water right holders or to facilitate aquatic habitat, diversion for recharge may be limited to periods when flows exceed the minimum instream requirements. Diversion may also be restricted during certain peak periods, if flows above a certain threshold are required for scour conditions favorable for generation of riparian habitat. Seasonal limits, which may be required during fish migrations or to ensure certain downstream water rights or requirements, also may further restrict diversion for recharge. These limitations may also affect diversions for surface water replacement of groundwater pumped for irrigation, but it is important to understand that the period during which river water can be diverted for recharge may not be substantially longer than would be required for surface water replacement during the irrigation season. This would need to be determined as part of a more detailed evaluation of aquifer recharge feasibility.

Evaluation of Recharge Potential

Well Drillers' Reports and Logs

Well drillers' reports and logs showed that a substantial thickness of fine-grained soil, primarily identified in the logs as clay, is found throughout the study area at or near the surface. The thickness varies from 2-3 feet to several tens of feet. In most parts of the study area, additional layers of fine-grained soils, including clay, silt, and volcanic ash, were identified at various depths in the vadose zone (area below the land surface and above the saturated zone). These

layers are frequently between 50 and 100 feet below the surface but often are at depths below 100 feet.

Well logs in some areas do not consistently identify laterally continuous subsurface layers of fines. These areas occur in the northern and northeastern part of the study area, primarily within a few miles north and south of the Highway 24 alignment northwest of the town of Minidoka and east of the Minidoka-Lincoln County line.

Almost all well logs in the study identified substantial thicknesses of hard, massive, unfractured basalt/lava rock in the vadose zone. The hard basalt flows occur at various depths from the surface to the water table. These basalt flows are considered potential barriers to vertical flow movement through the vadose zone and may represent an impediment to recharge using either surface recharge basins or vadose zone injection wells.

Previous Studies

Reclamation (2003) provided a brief summary of geologic and hydrogeologic conditions in and around the study area, including an explanation of the formation of subsurface lake sediments associated with impoundment of water and sediments behind an ancestral lava dam on the Snake River, as well as the presence of windblown, mostly fine-grained surface deposits across much of the study area, especially in the south and west parts.

Ralston (2008) showed from an analysis of well log data that much of the area is underlain by one or more layers of low-permeability sediments of varying thickness, as well as mostly fine-grained windblown surface deposits from a few feet thick up to several tens of feet thick. This report also found that the area in the north and northeast part of Unit B was largely free of subsurface sediment layers above the water table, and surface deposits of windblown fines are thinner and in some cases absent in the northern part of this unit.

Soils Maps and Survey Report

The NRCS soils maps (NRCS 1975) consulted for this study indicated that the subsurface conditions in the area around Highway 24 in the northeast part of the study area might be favorable for recharge infiltration basins. The soils maps identified the predominant soils to be silt loam. The specific soil types identified are provided in Appendix C.

Typical maximum infiltration rates for silt loam soils range from 0.27 to 0.4 inches per hour (Anderson 1998). At this low flow rate, a substantial amount of land would be required for recharge basins, as is discussed hereafter.

Soil Grab Samples

All soil grab samples obtained during the site visit (Refer to Site Visit Report – Appendix B) consisted of soils that were visually estimated to contain 80 to 90 percent silt or more, with only small percentages of fine sand and organic matter. This includes Sample 2, collected from the area that appears to have the greatest potential for surface recharge basins based on subsurface

conditions. Thus, the soil grab samples confirm the information provided on drillers' well logs and on NRCS soils maps for the area.

Snake River Flow Records

A preliminary review of instream flows on the Snake River at Minidoka Dam over the past 30 years (USGS 2008; Reclamation 2008b) indicates that the minimum daily mean instream flow rate is between 350 and 400 cfs, when available. These minimum flows usually are maintained from about mid-October to early April of each water year. The mean monthly flows for the same period indicate a low mean monthly flow rate of 2,840 cfs (November) and a high mean monthly flow rate of 12,860 cfs (May). From October to March, the mean monthly flow rate is between 2,840 cfs and 3,970 cfs. From April to early October, minimum flows generally far exceed 400 cfs and usually exceed 1,200 cfs from late April through September. Therefore it appears that diversion for recharge would be available for about six months per year in most years, and may be available year-round in many years. Wetter hydrological conditions and existing instream flow requirements may further expand or limit this recharge window. For the purposes of this evaluation, a six-month recharge window is assumed; however, a longer recharge period would reduce the area required for recharge as well as the sizes of pumps and pipelines required for conveyance, but might limit the amount of recharge that could be accomplished in years with prolonged low-flow conditions in the Snake River between October and April. Assuming that the objective would be to recharge approximately 225,000 ac-ft/yr, a recharge rate of 1,250 ac-ft/day, or approximately 630 cfs, would be required for a six-month recharge window. A six-month recharge program would presumably need to accommodate 630 cfs diversion and infiltration or injection. A longer recharge program, perhaps nine months (allowing for limited in-stream low-flow restrictions and for freezing temperatures) would reduce the recharge rate to about 420 cfs, or 833 ac-ft/day.

RECHARGE METHOD DESCRIPTIONS AND CONSIDERATIONS

General

Three primary methods of recharge have been considered for this study. These include recharge by means of shallow infiltration basins, injection into vadose zone wells, and aquifer injection wells. Each of these methods is discussed below.

OPTION 3a - Infiltration Basin Recharge

Of the recharge alternatives identified previously, infiltration basin recharge would be the most economical and practical for recharging large volumes of untreated surface water for irrigation use. The costs of the facilities would be lower because the pipeline distribution system would be reduced in size, permit requirements would be less rigorous, and treatment of raw surface water probably would not be required. Operation and maintenance costs would be less for surface infiltration basins than for the other recharge methods.

Surface recharge basins consist of large, shallow basins which are filled with recharge water to a depth of a few feet. Infiltration occurs through the soils underlying the basins. Often, recharge

basins are designed with a layer of clean, coarse sand on the floor of the basins. This facilitates basin maintenance because it is easier to disc the sand layer than most native soils. Periodic drying, discing and loosening of the basin floors would be necessary to reduce shallow soil compaction and prevent clogging. As the sand filters out fines in the recharge water, the top layer of sand can be scraped off from time to time and replaced with a clean layer. Typically, recharge basins are constructed in multiple cells, allowing variable rates of recharge without excessively loading any one cell. Multiple cells also allow for rotation of use, which makes it possible to periodically dry out cells for cleaning and maintenance. It is common to have about 50 to 70 percent of cells in use at a given time, with the remaining cells dry.

The surface and subsurface geologic requirements for effective shallow recharge basins include large areas of permeable soils at the surface, as well as minimal subsurface obstructions to vertical infiltration between the recharge basins and the water table. For this study area, subsurface obstructions would include silt or clay layers, unfractured/massive basalt flows, and volcanic ash, which can cause perching of groundwater and prevent or slow the vertical rate of movement to the water table. Primary soils types identified in the study area are shown in Table C1.

TABLE C1
SELECTED SOILS
A&B IRRIGATION DISTRICT

Soil Name	Description	Relative Permeability
Portneuf Silt Loam	Mixture of sand, clay, organics, and predominantly silt	Low to Moderate
Minidoka Silt Loam	Mixture of sand, clay, organics, and predominantly silt	Low to Moderate
Kimana Silt Loam	Mixture of sand, clay, organics, and predominantly silt	Low to Moderate
Portino Silt Loam	Mixture of sand, clay, organics, and predominantly silt	Low to Moderate
Wheeler Silt Loam	Mixture of sand, clay, organics, and predominantly silt	Low to Moderate
Rock Outcrop	Solid bedrock exposure	Low

Source: NRCS 1975.

The northeast part of the A&B study area would be upgradient of many wells in A&B and therefore may be suitable for recharge by means of vadose zone injection wells. Soils in this area also tend to be thinner than elsewhere, and fewer subsurface barriers such as fine-grained sediments and/or unfractured subsurface basalt flows are identified in well logs. However, some logs in this area do identify subsurface fine-grained soil and ash layers, which may indicate either that the low-permeability layers are laterally discontinuous, or they simply were not identified during well drilling.

The location or locations of surface recharge basins are also important relative to the areas intended for aquifer recharge. The recharge basins must be in reasonably close proximity to the

groundwater users, and must be either upgradient of all areas of pumping, or close enough downgradient that mounding of groundwater in the aquifer would allow local reversal of gradient or at least mitigate drawdown from upgradient pumping. Furthermore, except in the case of fractured or jointed bedrock, it is common for horizontal permeability to far exceed vertical permeability, so that lateral spreading of recharge water is likely to occur at basalt flow contacts or within soil layers and interbeds. Thus, the recharge water from surface infiltration basins may travel a considerable distance horizontally from the point of recharge before reaching the water table, which could cause some recharge water to migrate out of the service area and may reduce the available volume of recharge for the intended users.

If large volumes must be recharged, the infiltration basins may necessarily occupy a large surface area. The size of the basins footprint will depend on the permeability of the soils, the required rate of recharge, and shallow subsurface conditions that would affect saturation of the soils. Recharge basins are typically designed to be most effective at depths of only a few feet of recharge water to maximize the contact with the land surface and minimize soil compaction under the weight of the water column. Additional land is required to allow some recharge cells to be taken out of service for drying and maintenance; typically 1/3 to 1/2 of the recharge basin cells would be out of service at any one time. Unless the infiltration rate through the native soils is very high, the land footprint of the basins may occupy many acres, which may require taking cropland out of production for the basins. If large surface areas are required for recharge basins, the potential losses to evaporation may also be high during the warmer months.

Aquifer water quality would be subject to the requirements of IDAPA 58.01.11, *Ground Water Quality Rule*, which proscribes degradation of groundwater quality. Groundwater monitoring (monitoring wells and/or sampling of production wells) would be required to demonstrate compliance with this Rule. Recharge basins are regulated by the Idaho Department of Environmental Quality (DEQ). DEQ may require that a recharge basin program be governed by an individual program permit, which could regulate loading rates, recharge water quality, groundwater quality, monitoring requirements, and other features of a recharge basin program.

An estimate of the area required for recharge infiltration basins includes the following assumptions:

- Average infiltration rate of 0.3 inches/hour
- Recharge period of 6 months
- Constant recharge rate of 625 cfs (225,000 acre-feet over a 6 month period – Refer to Appendix C for additional discussion)
- 1/3 of all recharge cells would be out of service at any given time during operations for drying and maintenance
- 25 percent additional land area required for berms, roads, facilities, etc.

At a recharge rate of 625 cfs (=624 ac-in/hr), with an infiltration rate of 0.3 in/hr, a minimum of 2,080 acres of recharge basins would be required in service at any given time. Assuming this represents 2/3 of the basins with 1/3 out of service for maintenance, the required basin area would be 3,120 acres. Allowing an additional 25 percent of area for berms, roads, etc., the total estimated footprint would be approximately 3,900 acres. A cursory review of land prices in the

Burley-Rupert area suggests that irrigated land is currently selling for approximately $1,700 to $2,500 per acre. Assuming land values at $2,100 per acre, the cost of the real estate required for the surface recharge basins would be more than $8 million. This cost would be in addition to the costs for the intake structure, pump station, and pipelines necessary to convey water to the recharge basins.

If surface water from the Snake River can be diverted over a longer period than six months and if it is understood that in some years diversion limitations during low-flow conditions would reduce the total amount of annual recharge, a smaller recharge basin area could be used. For example, if a nine-month recharge period were used rather than a six-month period, the footprint would be approximately 2,600 acres rather than 3,900 acres. Further capital savings would be realized by reduction of pipe and pump sizes. Operation costs would increase but the area required for maintenance would decrease, so the net O&M differences would be minimal.

As noted, surface soils throughout the evaluated portion of the study area are primarily high in windblown deposits of silt. Silt is a fine-grained material and is generally low in permeability. Therefore, the NRCS soils map indicates that the area where subsurface conditions might be more favorable to recharge infiltration basins (near Highway 24 in the northeast portion of A&B, as determined from evaluation of well logs), is overlain by a veneer of low-permeability soils that would have to be removed before effective infiltration could occur. However, since the well logs indicate that in most areas the surface soils are several feet deep, this would probably be impractical because of the large volume of soil that would have to be excavated.

It should also be noted that if recharge is restricted to one area in the north-northeastern part of Unit B of A&B, the benefits of a successful recharge program would be realized primarily in the eastern part of Unit B, with less benefit further west. Basins could be distributed more widely across the site, but the presence of subsurface vertical flow barriers throughout much of the area probably limits this option. Furthermore, the cost savings associated with recharge would be minimal if the conveyance system is required to distribute water to multiple locations.

Some perching of groundwater occurs on the subsurface layers of fines. Basalt is highly permeable at fractured and rubbly flow contacts but may be much less permeable to vertical groundwater infiltration where massive, unfractured basalt flows exist. Vertical flow migration can require a long and tortuous path to the aquifer while finding passage around these vertical flow impediments.

OPTION 3b - Vadose Zone Injection Wells

Vadose zone injection wells would consist of multiple shallow wells, designed to convey surface water to a given depth below ground surface but above the water table. The rationale for vadose zone injection is primarily to allow recharge in areas where surface soils are unsuitable for surface recharge basins, either because of low permeability or limited availability of surface area. Vadose zone recharge also reduces evaporation losses and visual impacts associated with recharge basins and can reduce the possibility of stability or seepage problems stemming from saturation of shallow soils around the recharge basin, especially if perching of recharge water is likely. Vadose zone wells typically are designed to extend below fine-grained shallow soils or

unfractured bedrock, or may target shallow fault zones under certain conditions. Vadose zone wells are typically less expensive to construct and maintain than recharge wells that extend into the saturated zone. If the distance from the depth of recharge to the water table is long enough and the recharge pathway passes through granular materials (e.g. sands), filtration benefits may be realized. Significant filtration is unlikely to occur if the vertical migration pathway is through fractured bedrock rather than granular media.

As with recharge basins, vadose zone injection wells would be placed upgradient of production wells. Vadose zone injection wells have an advantage over recharge basins in that they can be targeted throughout the upgradient part of the wellfield, or alternatively in clusters; if dispersed widely, however, much of the capital cost savings of reduced pipeline footage would be lost when compared to Alternative 2.

Drillers' reports for many of the production wells throughout the study area indicate that the aquifer includes multiple zones of high hydraulic conductivity and high well production capacity. This is an indicator that aquifer recharge injection wells may be technically feasible. For high rates or large volumes of recharge, vadose zone injection may require many wells. The recharge depths should target zones of higher hydraulic conductivity to minimize the generation of head in the well and to keep the number of required wells as low as possible. Within the study area, the zones of recharge would most likely correspond to basalt flow contacts where rubbly basalts can provide substantial secondary porosity if not filled with sediments or precipitates. Cinder zones also often can be highly permeable and may be good injection zones as well.

Vadose zone injection requires that wells be designed in such a way to control down-well generation of head to prevent excessive backpressure in the recharge system. On the other hand, well injection water should not be allowed to free-fall or cascade down the wells, because this results in entrained air in the recharge water. Entrained air in recharge water promotes biofouling in well screens and perforations, and can cause airlocking in soil interstitial spaces. These problems can greatly reduce the receiving capacity of the recharge zone and can be difficult to resolve. Preventing cascading in injection wells is usually accomplished by using specially designed downhole injection valves that control the rate of injection and keep the well column full of water. These valves are expensive, adding to the cost of each well and further necessitating the need to minimize the number of recharge wells.

Injection wells also often require special construction materials, including stainless steel screen and casing, to prevent either excessive corrosion or to prevent flushing of iron from the casing into the formation. This latter problem can promote precipitation of iron oxides in the aquifer formation, which can clog the formation and reduce aquifer recharge capacity.

Most injection wells are designed with filter packs, even in bedrock. This is done to prevent forcing sediments in injection water out into the formation. For injection wells and dual wells (i.e. Aquifer Storage and Recovery, or ASR, wells), filter packs can be flushed by reversing the rate of flow, or in the case of injection-only wells, swabbing and bailing during periodic maintenance to remove sediments captured in the filter pack. This may not be necessary if injection water is exceptionally clean, such as treated potable water.

Recharge using vadose zone injection wells would be subject to the requirements of the Underground Injection Control (UIC) program component of the Federal Safe Drinking Water Act, which in Idaho is administered by the Idaho Department of Water Resources (IDWR) as the governing agency. The Idaho UIC program is subject to the requirements of Idaho Administrative Rule 37.03.03, *Rules and Administrative Standards for the Construction and Use of Injection Wells*. Under Idaho's UIC program, aquifer recharge wells are classified as Class V, Subclass 5R21. The terms for permitting of aquifer recharge wells, including vadose zone injection wells, is required under Sections 37.03.03.030 to 050. The injection program must comply with nondegradation requirements of groundwater quality as specified under Section 200 of IDAPA 58.01.11, *Ground Water Quality Rule*. Section 200 refers to numerical groundwater quality standards; however, IDAPA 58.01.301.02 restricts activities that have the potential to degrade existing groundwater quality, even if numerical standards are not exceeded. This may occur in many instances where surface water is used to recharge groundwater. Provision is made for this in Section 400 by implementation of Best Management Practices (BMPs). In most instances for recharge wells under the UIC program, compliance with this requirement is achieved by conventional water treatment (filtration and disinfection) prior to injection.

If chlorination is used for disinfection, it may be necessary to determine whether disinfection byproducts (DBPs) will occur and, if so, whether they will need to be treated before injection can occur. Some DBPs are removed in the natural environment, but whereas some DBPs are removed under aerobic conditions, others require anaerobic conditions for removal. Thus, natural conditions favorable for removal of some DBPs may not be favorable for removal of other DBPs.

OPTION 3c - Aquifer Recharge Injection Wells

Aquifer recharge via injection wells would be similar to vadose zone injection wells, except that the wells would extend to one or more saturated zones of the production aquifer. The wells would be open to flow contacts or cinder layers where hydraulic conductivities are high. For the purposes of this study, ASR wells (as opposed to injection wells) are not considered as part of this option because an existing network of production wells is already in place.

Design considerations for aquifer recharge injection wells are similar to those of vadose zone injection wells. Because they would be much deeper than vadose zone injection wells, the risk of cascading is greater, so it is essential that cascading be controlled by means of downhole valves. Injection wells must be maintained regularly to prevent or mitigate biofouling, scaling, corrosion, sediment buildup in the well bottom or formation, and other concerns. Periodic rehabilitation is required to extend the life of the wells.

Because aquifer recharge injection wells are somewhat deeper than vadose zone injection wells, they would be more expensive to construct. Maintenance and rehabilitation also may be more expensive, although it may be easier to flush out solids during rehabilitation because the wells can produce groundwater for flushing rather than losing injected flush water to the formation.

As with vadose zone injection wells, aquifer recharge injection wells would be subject to the UIC program, including the requirements of Idaho Administrative Rule 37.03.03, *Rules and*

Administrative Standards for the Construction and Use of Injection Wells, as well as IDAPA 58.01.11, *Ground Water Quality Rule*. As with vadose zone injection wells, aquifer recharge wells are classified as Class V, Subclass 5R21. The terms for permitting of aquifer recharge wells, including vadose zone injection wells, is required under Sections 37.03.03.030 to 050. Because aquifer recharge injection wells put surface water directly into the aquifer, they do not provide the benefit of natural filtration prior to coming in contact with groundwater. Therefore the risk of introducing lower-quality water directly into groundwater is greater, and the need for treatment prior to recharge is greater.

OTHER OPTIONS

Two other options for the A&B area were considered but not evaluated. These include partial replacement of the groundwater system by surface water, and conjunctive use. Partial replacement would consist of replacing only part of Unit B's groundwater pumping system with surface water, leaving the remainder to be irrigated by groundwater pumping as is currently done. A conjunctive use system would consist of a surface water system distributed throughout Unit B and would be the primary irrigation water source, but would be sized for a lower capacity and would be supplemented by the existing groundwater pumping system during periods of high demand. These options are discussed briefly in the Recommendations section of this report. Additional discussion of the feasibility of these options is provided below.

Previous work by Reclamation (2003) presented information developed by the Idaho Water Resources Research Institute (IWRRI) with regard to net recharge and discharge to and from the Eastern Snake River Plain. The IWRRI research estimated that the area in and around A&B had a net recharge (after accounting for evapotranspiration (ET) and other factors) of 0 to 0.031 ft/day in July 2001 on land where surface water irrigation was occurring, attributable to canal seepage and infiltration of irrigation water. A net discharge of 0 to 0.029 feet per day was estimated on land where groundwater irrigation was occurring, which was attributed entirely to groundwater pumping. These rates were assumed to be reasonably representative of recharge and discharge occurrences during the normal irrigation season. Similarly, most of the area was found to have a net recharge of 0 to 0.0013 feet per day in March 2000, which was assumed to be representative of the non-irrigation season recharge associated with precipitation infiltration and low ET.

The findings of the Reclamation study suggest that some recharge from surface irrigation does occur in the A&B area. A partial-replacement program whereby surface water replacement is used on part of the area while the remaining part continues to pump groundwater, may include a certain amount of incidental recharge from surface water irrigation. Because the estimated net rate of incidental surface water recharge from irrigation is close to the estimated net rate of groundwater discharge, it may be possible to configure a partial-replacement system that has a net balance of groundwater discharge and recharge within the A&B area. Similarly, a conjunctive-use system whereby surface water is distributed throughout Unit B but is supplemented by groundwater pumping during peak demand, may result in a groundwater recharge rate that replaces most or all of the groundwater pumped over the period of the irrigation season.

Appendix D

APPENDIX D

DATA SOURCES

Well Driller Reports/Well Logs

An evaluation of 218 well driller reports and associated lithologic logs for 177 wells (some logs are for different depths of the same well due to deepening) within the study area, provided electronically by IWRB, included a review of the following conditions as reported in the logs:

- Well location
- Occurrence of fine-grained soils, hardpan, or volcanic ash layers between the ground surface and the water table
- Occurrence of hard, massive basalt that was not identified as fractured or otherwise demonstrating secondary permeability
- Presence of fractured bedrock or layers of rubbly, broken basalt beds or cinders
- Presence of sands, gravels, or cobbles
- Recorded pumping rate and associated drawdown at time of construction.

Well Location. The location of the wells was noted within the study area. Because the groundwater flow direction in the study area is west to southwest, logs for wells in the north and northeast part of the study area were considered more indicative of conditions for the vicinity where most recharge would have to occur to effectively be used by production wells in the A&B Irrigation District. However, logs for wells throughout the study area were examined to determine where aquifer recharge might be feasible.

Fine-Grained Soils. The depths and thicknesses of layers of fine-grained soils, accumulation of cemented soils (also known as hardpan or caliche), or volcanic ash layers were noted for each log from the ground surface to the water table. Very minor layers of fines were disregarded if noted; however, since most common drilling methods preclude detection of very thin layers of fines in drill cuttings, in most instances it was assumed that if the driller's log included a note on fines, the layer was thick enough to inhibit vertical migration of fluids.

Hard, Massive Basalt Flows. Although fractured or rubbly basalt layers can be highly permeable and vertical (columnar) jointing can allow substantial vertical downward migration of groundwater in the vadose zone, massive, hard, unfractured basalt flows act as vertical flow barriers. Therefore if the driller's log indicated the presence of "hard", "firm", or "unfractured" basalt or lava rock in the vadose zone, it was assumed that the flow is massive and would act as a vertical flow barrier. Although drillers' descriptions of geologic conditions are often brief or incomplete and sometimes inaccurate, it was assumed that if a driller noted this condition, he was experiencing difficulty drilling through it and therefore the basalt flow is probably a vertical flow barrier or inhibitor.

Fractured Rock. Driller reports of fractured, rubbly, "creviced", "broken", or "soft" rock (in this instance, almost always basalt/lava rock) or cinder layers were interpreted to mean that these layers are possible avenues of groundwater movement. Because in most logs such layers appear

to be relatively short intervals, they were most often assumed to represent horizontal rather than vertical avenues of flow. Such zones may be suitable for vadose zone recharge, but unless underlain by more fractured rock or granular soils, they may not be suitable for recharge target zones if downward flow is restricted.

Sand and Gravel Layers. Where present, sand and gravel layers represent potential target zones for vadose zone recharge wells. As with fractured or rubbly rock or cinders, however, recharge to these layers in the vadose zone may not be an effective way of recharging the aquifer unless underlain by fractured rock, and are in any event infrequently found in the study area.

Pumping Rates and Drawdown. Pumping rates and, normally, associated water level drawdown were recorded on most driller reports at the time of pump installation. Although these records may no longer be entirely representative of current conditions, they provide an indication of production capacities in the wells. A high pumping rate with a low drawdown is an indication of a high production capacity, whereas a higher drawdown or, alternatively, lower pumping rate, are indicators that a well is less productive. Well productivity and corresponding drawdown is an important indicator of aquifer characteristics within the open or screened intervals of a well.

Previous Studies

Two previous studies were consulted as part of the aquifer recharge feasibility evaluation. These include a Reclamation groundwater model simulating the impacts groundwater pumping in an adjacent area on groundwater conditions in the A&B Irrigation District (Reclamation 2003), as well as a hydrogeologic analysis of the area in and around the A&B Irrigation District (Ralston 2008). The Reclamation report included a simulation of recharge and discharge and an estimate of surface recharge rates relative to discharge from pumping. The Ralston report included an evaluation of information from the drillers' reports and well logs that was used to corroborate the findings of the well log review.

Soils Maps and Survey Reports

Surface soil conditions are a critical component of any recharge program associated with recharge infiltration basins. Deep, fine-grained surface soils are much less favorable for infiltration basin placement than coarser, granular soils.

Soil surveys have been conducted by the Natural Resources Conservation Service (NRCS), formerly known as the Soil Conservation Service (SCS). A soil survey of the study area is included in an NRCS report (NRCS 1975). The survey report includes soils maps showing the locations of various surface soils of the study area, as well as descriptions of the soils and their characteristics. To streamline this evaluation, soils maps were reviewed primarily for areas where subsurface conditions (as determined from the well driller reports/logs review) did not automatically preclude placement of recharge infiltration basins. A list of soil types in those areas was compiled, and the relevant characteristics (amount of fines, relative permeability) were noted.

Soil Grab Samples

During an on-site visit to the study area, six surface soil grab samples were collected from locations throughout the site. Because private property access had not been granted for the study area, soils were collected along roadsides rather than from fields. Care was taken to collect soil samples from natural soils, rather than roadside fill. The locations of sample locations are shown in the Site Visit Report (Refer to Appendix B).

Soil grab samples were visually examined to qualitatively determine whether they would be suitable for infiltration basins, based on relative content of fines (silt and clay) compared to coarser soils (sand and gravel). No attempt was made to quantify grain size content by sieve analyses or other means, rather, soil samples were used to verify soils maps provided in the NRCS survey report.

Appendix E

APPENDIX E

DISCUSSION OF INTERNATIONAL CLASS 5 OPINION OF PROBABLE COST

In presenting this cost opinion, there are several factors and conditions that should be noted as they serve as the basis of this opinion. The factors include price increases and volatility, cost escalation, conditions and accuracy of the opinion, and assumptions, exclusions, and contingency for the opinion. A discussion of each of these factors is presented in the following sections.

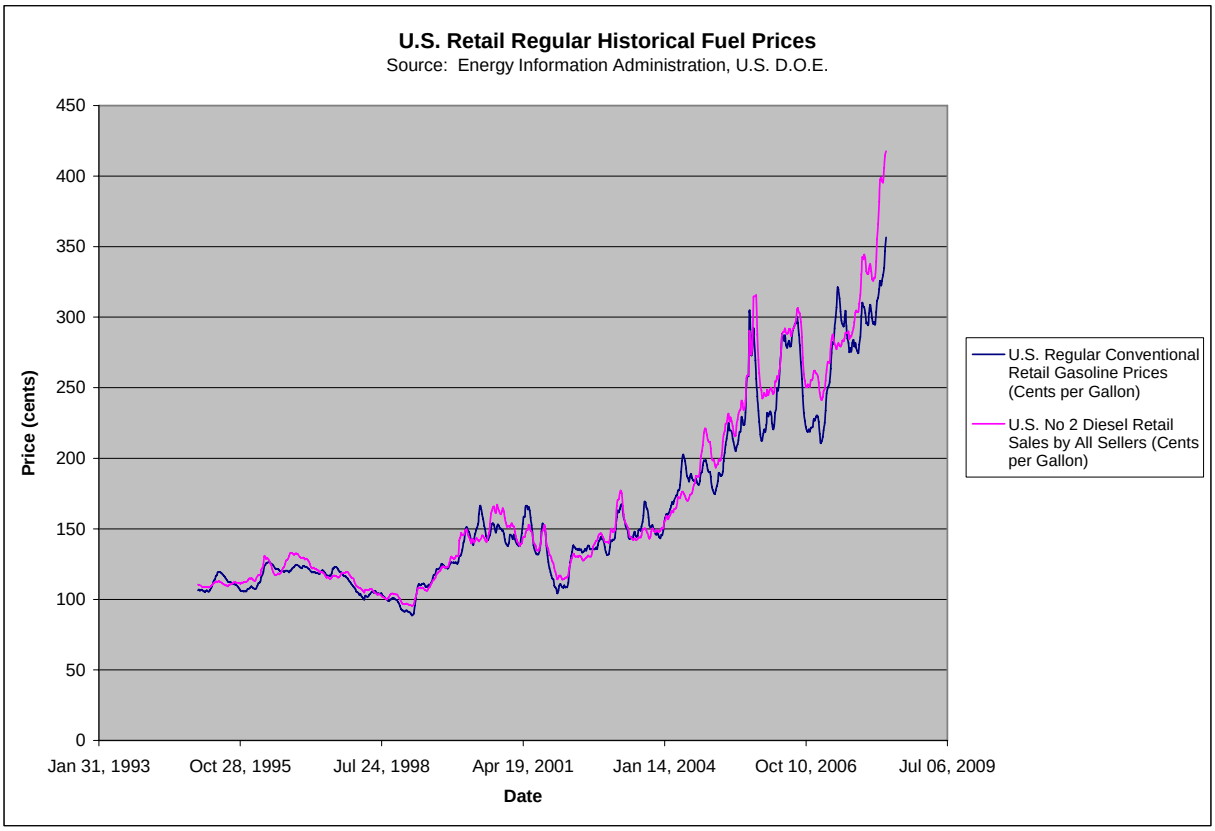
Price Increases and Market Volatility

In recent years, Hurricane Katrina devastated significant portions of the United States, China has undertaken a major expansion of their infrastructure, and most recently, the cost of fuel has been rising at an ever-increasing rate. Occurrences such as these, both abroad and in the states, have an impact on the cost of fuel and materials, which in turn has an impact on the cost of labor, materials and equipment for construction projects.

Fuel Prices. The construction of pipelines and pump stations is very fuel intensive. Materials delivery equipment and construction equipment are all powered by fuel. The increasing fuel prices results in increasing construction costs. The increase in fuel prices over the last five years is shown in Figure E.1.

FIGURE E.1

HISTORICAL FUEL PRICES

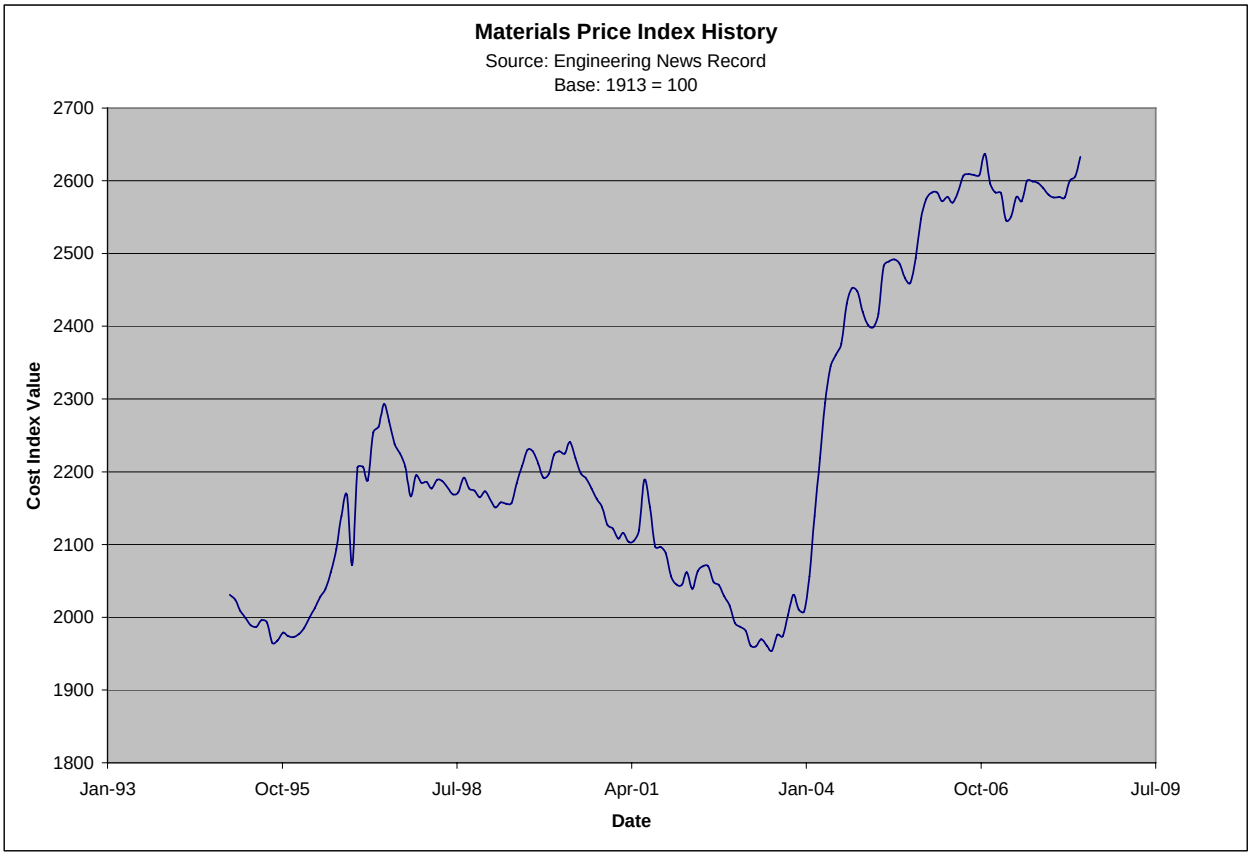


Steel Prices. The construction of pipeline and pump stations is very steel intensive. Steel is used for the manufacturing of pipe, pumps, valves, and reinforcing steel for the concrete structures. Through recent correspondance with American, a steel pipe manufacturer, it was noted that steel prices have increased 20 to 45 percent within the last 6 months. American suggested the opinion be reevaluated periodically due to current metal market volatility.

Materials Prices. Another indicator of the changes in the cost of materials is the Engineering News Record (ENR) Materials Price Index. The Materials Cost Index is the materials component of ENR's building and construction cost indexes. It tracks the weighted price movement of structural steel, portland cement and 2 × 4 lumber. All of these materials are used in the construction of pipelines, pumping stations and hydropower facilities. The historical ENR Materials Price Index is shown in Figure E.2.

FIGURE E.2

HISTORICAL ENR MATERIALS PRICE INDEX

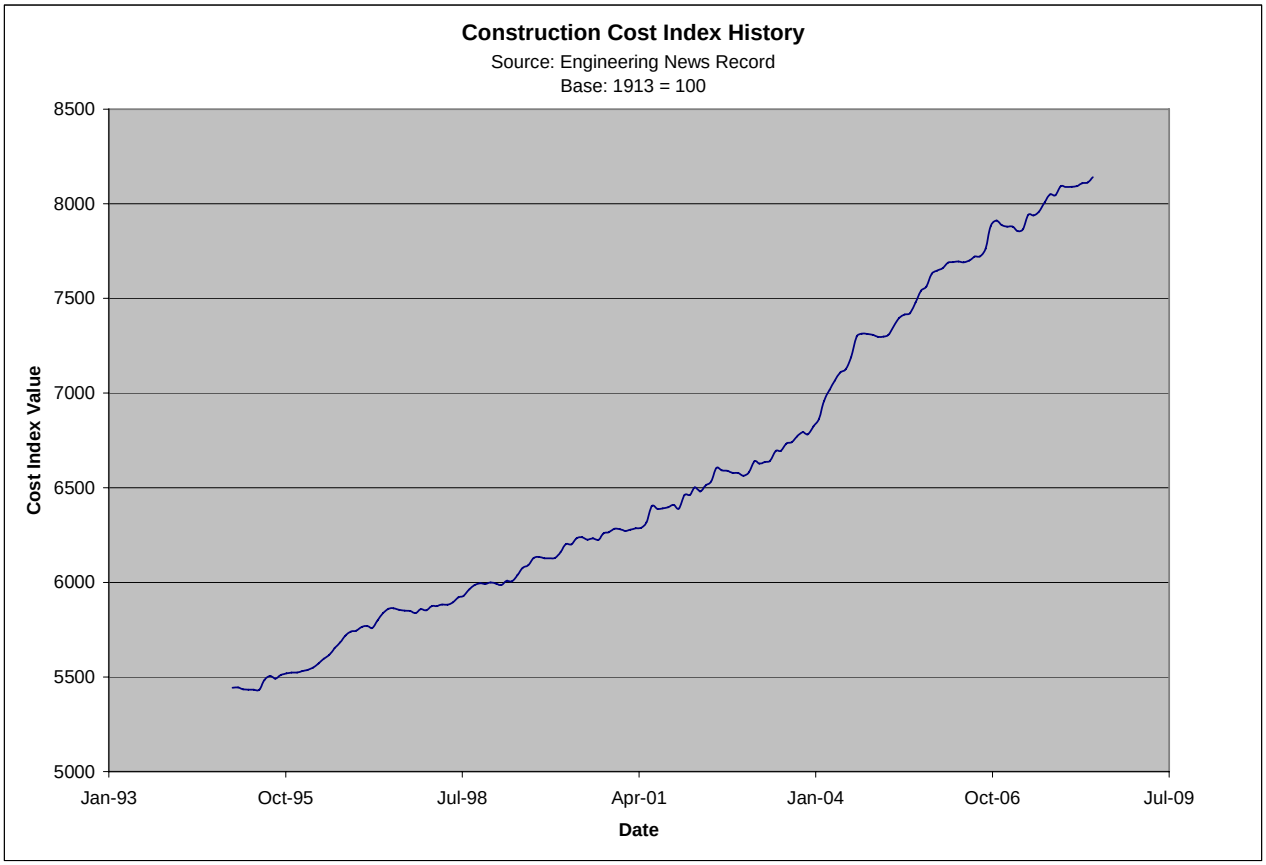


Inspection of the cost curve reveals a significant change in the rate of increase of costs since January 2004.

Construction Prices. The increases in fuel and materials have increased construction costs. A gauge of the impact on these items can be determined through the use of ENR's Construction Cost Index (CCI). The ENR CCI is a commonly used reference point for cost opinions and represents 200 hours of common labor at the 20-city average of common labor rates, plus 25 cwt of standard structural steel shapes at the mill price prior to 1996 and the fabricated 20-city price from 1996, plus 1.128 tons of portland cement at the 20-city price, plus 1,088 board-ft of 2 × 4 lumber at the 20-city price. The historical ENR CCI values are shown in Figure E.3.

FIGURE E.3

HISTORICAL ENR CCI VALUES



Inspection of Figure E.3 reveals a change in the slope of the cost curve at January 2004. The slope of the curve from January 2004 to present day is more than double the slope of the curve from 1995 to January 2004. The implication of this doubling is that construction costs are increasing at a rate double that occurring prior to 2004.

Construction of pipelines and pump stations is fuel and steel intensive, therefore the ENR CCI is believed to underestimate the true impact of changes in the cost of these items on overall costs. The CCI would tend to dampen the extreme price increases.

Escalation of Costs

An escalation factor is a projection of future inflation rates and price increases. At the present time no escalation has been assumed for this project. However, given that the construction of the project will occur sometime in the future, an escalation factor will need to be applied when considering construction costs at this later point in time.

Conditions of the Opinion of Probable Cost

The cost opinion presented in this report has been prepared in accordance with the cost estimate classes defined by the Association for the Advancement of Cost Engineering.

MWH has no control over costs of labor, materials, competitive bidding environments and procedures, unidentified field conditions, financial and/or market conditions, or other factors likely to affect the cost opinion of this project, all of which are and will unavoidably remain in a state of change, especially in light of the high volatility of the market attributable to Acts of God and other market events beyond the control of the parties. The cost opinion is a "snapshot in time" and the reliability of this opinion will inherently degrade over time. MWH cannot and does not make any warranty, promise, guarantee, or representation, either express or implied, that proposals, bids, project construction costs, project capital costs, or cost of operation or maintenance will not vary substantially from MWH's good faith cost opinion.

Accuracy of the Probable Opinion of Cost

The Association for the Advancement of Cost Engineering (AACE) has established different classes of cost estimates to correspond with different levels of project definition, expected accuracy range, and preparation effort. The definition of the International Class 5 cost opinion used in this report is described below.

- **The AACE International CLASS 5 Cost Estimate –** Class 5 estimates are generally prepared based on very limited information, and subsequently have wide accuracy ranges. Typically, engineering is from 2 percent to 10 percent complete. They are often prepared for strategic planning purposes, market studies, assessment of viability, project location studies, and long range capital planning. Virtually all Class 5 estimates use stochastic estimating methods such as cost curves, capacity factors, and other parametric techniques. Expected accuracy ranges are from –20 percent to –50 percent on the low side and +30 percent to +100 percent on the high side, depending on technological complexity of the project, appropriate reference information, and the inclusion of an appropriate contingency determination. Ranges could exceed those shown in unusual circumstances.

Cost Opinion Assumptions, Basis, Exclusions, Contingency, Engineering

Assumptions. The following construction assumptions were made in preparation of this cost opinion.

- No groundwater is encountered during pipe line construction

- No shoring or blasting is required during pipeline trench excavation
- Excavated material is suitable for backfill
- Pump stations are basic in nature
- Operation and maintenance easements, as opposed to land acquisition, will be required for pipeline construction

Basis. The cost opinion of the pressurized distribution system was based on a conceptual layout of OPTION 2b of Conceptual Alternative 2 as described in the report text. To meet Class 150 requirements and avoid higher costs associated with higher pressure classes, all pipeline pressures were kept at or below 150 psi. A minimum pressure of 10 psi was used for a lower desirable pressure limit. Based on previous experience, a velocity of 6.6 ft/s was chosen as providing a good balance between pump station horsepower and pipe line diameter. Given the 150 psi criteria, no booster pump stations were required for this system. Therefore, the evaluated distribution system had one main pump station sized to pump flow throughout the entire system.

Although this single pump station provided a basis for the conceptual analysis, it is not the most efficient set up, producing unnecessarily high heads in the vicinity of the pump station. A more appropriate configuration would be to locate booster pump stations along the main lines thereby better matching supply head with demand discharge pressures. Some considerations with this configuration include additional capital costs associated with a more extensive SCADA system and additional power transmission to the booster pump stations. However, the size of the main pump station would be reduced dramatically and the annual power usage would be reduced by optimizing the system's in-line and discharge pressures.

For determining pipeline unit costs, PVC was used for pipe diameters less than 24-inch; Cement mortar-lined, lap-welded steel pipe was used for pipe diameters 24-inch and greater. Installed pipe costs included tape wrapping per AWWA C214, a crown cover of 3 feet, excavation, placement, lap welding, compaction, site restoration, unidentified costs such as wet conditions, two AR and two blow-off valves per mile of pipe length, cathodic bonding for gasketed pipe under 66-inch and welded pipe bond for pipe greater than 66-inch.

Unit costs for the opinion included labor burden, small tools and equipment, contractor field, and contractor overhead and profit as described below. Davis-Bacon prevailing wage rates were used.

- Labor Burden – This includes payroll taxes and Workman's Compensation Insurance. This is applied only to the labor cost category. Assumed for this opinion: 32%.
- Small Tools and Equipment – This includes an allowance to cover the cost of small tools, equipment, and consumables. This is typically calculated as a percentage of the labor cost category. Assumed for this opinion: 5%.

- Contractor Field Overhead – This includes costs that would be incurred by the performing contractor to pay for general requirements and field overhead. This includes costs for Builder's Risk insurance, General Liability insurance, and payment and performance bond purchased by the general contractor or construction manager. Assumed for this opinion: 5%.
- Contractor Overhead and Profit – This includes contractor's home office overhead, and profit. Assumed for this opinion: 10%.

Exclusions. Sales tax was excluded from the cost opinion because this project was assumed to be completed by a tax-exempt agency.

Contingency. A project cost contingency is provided to cover unknowns and changes in project definition that take place as a normal course of action as the project proceeds from conceptual to final design. Potential unknowns that could result in unforseen additional costs include blasting of any rock encountered during excavation, dewatering, unforeseen field conditions, bid climate, and number of bidders. For this Class 5 opinion, two contingencies were included or considered as described below.

- Unlisted Items Contingency – This includes errors and omissions in the opinion including the variability associated with the quantification effort and the misinterpretation of design documents leading to substantial quantity differences. Assumed for this opinion: 15 %.
- Scope Contingency – This includes scope insufficiencies that could occur due to project requirement oversights, omissions, misinterpretations, and undefined regulatory considerations. Considered for this opinion: 20%.

Engineering, Administration, Environmental, Services During Construction (SDC). Engineering, administration, environmental and SDC costs include costs for construction contingency, program management, pre-design planning, environmental planning and review, engineering design, construction management, and department and agency fees.

International Class 5 Opinion of Probable Cost Summary

A summary of the Class 5 cost opinion is shown in Table E.1. The first part is an opinion of the construction cost with contingency followed by an opinion of the potential project cost. Final project costs presented in the cost opinion include the cost of construction, engineering, administration, environmental, services during construction and a contingency.

TABLE E.1

INTERNATIONAL CLASS 5 OPINION OF PROBABLE COST

	Quantity	Unit	Unit Cost	Unit	Total
001 Minidoka North Side Canal Modifications	34,815	CY	10	/CY	346,162
002 Bridges Demo and Rebuild 65' long by 25' wide	2	EA	386,115	/EA	772,231
003 Pump Station 518	1	EA	16,571,475	/EA	16,571,475
004 Pump Station 482	1	EA	15,245,757	/EA	15,245,757
005 Pipeline 16"	275,880	LF	53	/LF	14,629,563
006 Pipeline 20"	101,380	LF	66	/LF	6,720,064
007 Pipeline 30"	101,640	LF	119	/LF	12,127,138
008 Pipeline 36"	39,600	LF	159	/LF	6,299,812
009 Pipeline 42"	13,470	LF	199	/LF	2,678,613
010 Pipeline 48"	4,230	LF	232	/LF	981,363
011 Pipeline 54"	5,280	LF	285	/LF	1,504,955
012 Pipeline 60"	11,620	LF	378	/LF	4,390,380
013 Pipeline 64"	7,400	LF	477	/LF	3,531,713
014 Pipeline 66"	7,400	LF	504	/LF	3,727,919
015 Pipeline 72"	8,450	LF	524	/LF	4,424,915
016 Pipeline 78"	10,560	LF	603	/LF	6,369,810
017 Pipeline 84"	22,180	LF	689	/LF	15,290,301
018 Pipeline 90"	10,560	LF	782	/LF	8,259,753
019 Pipeline 96"	10,560	LF	875	/LF	9,239,724
020 Pipeline 102"	12,680	LF	1,021	/LF	12,943,780
021 Pipeline 108"	22,180	LF	1,107	/LF	24,552,694
022 Pipeline 114"	35,910	LF	1,246	/LF	44,750,140
023 Pipeline 120"	61,250	LF	1,372	/LF	84,042,233
024 14" RR Borings	200	LF	464	/LF	92,800
025 30" RR Borings	400	LF	901	/LF	360,595
026 102" RR Borings	200	LF	3,122	/LF	624,413
027 114" RR Boring	200	LF	3,566	/LF	713,236
028 16" BF Valve	25	EA	1,273	/EA	31,817
029 20" BF Valve	10	EA	1,955	/EA	19,554
030 30" BF Valve	15	EA	5,316	/EA	79,742
031 36" BF Valve	5	EA	8,259	/EA	41,296
032 42" BF Valve	2	EA	15,975	/EA	31,950
033 SCADA Pump St	2	EA	9,114	/EA	18,229
034 SCADA Main	1	EA	182,286	/EA	182,286
035 Substation	1	EA	911,431	/EA	911,431
036 Transmission	0.76	MI	729,145	/MI	554,150
037 Land Purchase Costs from Report	64	AC	1,525	/AC	97,573
038 Excavation for Regulation Storage	235,760	CY	7	/CY	1,562,756
039 Fill and Compaction for Regulation Storage	235,760	CY	3	/CY	781,378
Subtotal					305,503,701
040 Unlisted Items Contingency	15	%			45,825,555
BASE CONSTRUCTION COST					351,329,256
041 Engineering, Administration, Environmental,SDC	1	LS	10,000,000	/LS	10,000,000
Opinion of Probable Project Cost					361,329,256
ROUNDED OPINION OF PROBABLE PROJECT COST					360,000,000

The AACE International CLASS 5 Cost Estimate - *Class 5 estimates are generally prepared based on very limited information, and subsequently have wide accuracy ranges. Typically, engineering is from 2 percent to 10 percent complete. They are often prepared for strategic planning purposes, market studies, assessment of viability, project location studies, and long range capital planning. Virtually all Class 5 estimates use stochastic estimating methods such as cost curves, capacity factors, and other parametric techniques. Expected accuracy ranges are from –20 percent to –50 percent on the low side and +30 percent to +100 percent on the high side, depending on technological complexity of the project, appropriate reference information, and the inclusion of an appropriate contingency determination. Ranges could exceed those shown in unusual circumstances.*

Appendix F

Power Account - Usage from USBR Billing Summaries - A&B Power Cost.xls 1/22/2008

Year	Unit B Annual Amount Pumped (ac-ft)	Total Annual Power Used (kwh)	Power Per Amount Pumped (kwh/ac-ft)	1969-1979 Ave. kwh/ac-ft	Diff. from ave. 1969-1979 and post-1980 kwh/ac-ft	Total Increase in kwhs for Water Pumped	Price Per kwh from USBR (mills)	2008 Inflated Price per kwh from USBR (mills)	Increased $ since 1980 from Declining Ground Water Levels (Assuming 2008 Power Rate)
1969	222,290	93,176,480	419						
1970	195,268	84,076,573	431						
1971	204,486	81,598,856	399						
1972	216,578	90,286,299	417						
1973	211,035	86,435,152	410						
1974									
1975									
1976	192,574	79,650,905	414						
1977	219,136	91,620,040	418						
1978									
1979				415					
1980	193,026	83,757,875	434		19	3,598,510	4.0	9.66	$34,771
1981	215,057	88,146,810	410				5.1	11.94	
1982	178,645	76,912,067	431		15	2,724,811	5.1	11.57	$31,519
1983	177,069	76,363,979	431		16	2,831,325	5.1	11.21	$31,736
1984	182,390	78,741,010	432		16	2,998,622	5.1	10.86	$32,569
1985	194,690	82,138,282	422		7	1,287,769	5.1	10.52	$13,553
1986	186,998	78,136,904	418		3	480,708	5.1	10.20	$4,902
1987	209,647	89,143,265	425		10	2,081,693	7.7	15.00	$31,220
1988	212,409	86,725,593	408				7.7	14.53	
1989	215,241	89,285,426	415				7.7	14.08	
1990	211,532	98,061,266	464		48	10,216,687	7.7	13.65	$139,407
1991	190,024	85,523,615	450		35	6,610,789	10.6	18.16	$120,044
1992	213,797	100,967,007	472		57	12,181,782	10.6	17.60	$214,348
1993	174,193	82,221,148	472		57	9,882,543	10.6	17.05	$168,499
1994	205,813	95,764,022	465		50	10,294,541	10.6	16.52	$170,081
1995	160,552	77,426,767	482		67	10,753,173	10.6	16.01	$172,150
1996	189,953	88,262,865	465		49	9,379,731	10.6	15.51	$145,506
1997	174,267	83,185,343	477		62	10,816,215	12.7	17.96	$194,248
1998	160,254	76,080,611	475		59	9,530,770	12.7	17.40	$165,855
1999	175,018	82,272,797	470		55	9,591,546	12.7	16.86	$161,737
2000	207,090	95,300,601	460		45	9,300,810	12.7	16.34	$151,971
2001	196,367	89,387,698	455		40	7,840,848	12.7	15.83	$124,144
2002	182,635	84,385,248	462		47	8,540,826	15.6	18.85	$160,954
2003	183,153	83,394,968	455		40	7,335,723	15.6	18.26	$133,957
2004	181,921	86,329,178	475		59	10,781,389	15.6	17.69	$190,774
2005	150,163	71,296,751	475		60	8,937,437	19.3	21.26	$189,980
2006	172,525	81,150,661	470		55	9,504,697	19.3	20.60	$195,774
2007	183,834	89,694,337	488		73	13,352,122	19.3	19.96	$266,493
				Total 1980-2007					**$3,246,193**

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